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# AQUIFER CHARACTERISTICS AND GROUND-WATER MOVEMENT AT HANFORD

W. H. BIERSCHENK

JUNE 9, 1959



HANFORD LABORATORIES

HANFORD ATOMIC PRODUCTS OPERATION
RICHLAND, WASHINGTON

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Chemical Research and Development Operation


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# AQUIFER CHARACTERISTICS AND GROUND-WATER MOVEMENT AT HANFORD

## INTRODUCTION

In proper environmental situations low and intermediate level radioactive wastes are susceptible to either direct dispersal to the environment or discharge following treatment. At Hanford an estimated 3.8 billion gallons of intermediate-level waste containing about 2,500,000 gross beta curies have been discharged to ground since startup in 1944 through December, 1958. In addition to these wastes, approximately 35 billion gallons of normally uncontaminated process cooling water has been discharged to open ponds or swamps.[1] Such disposal of large volume, low-level waste to the ground has proved to be effectual and economical - principally because of the environmental conditions at the Hanford site.

At Hanford, the semi-arid climate, the permeable surficial sediments, and the deep water table combine to produce a situation wherein most of the radioactive materials in the waste are trapped by electro-chemical bonds in the sediments as the waste percolates down to the water table. Those wastes that reach the water table move with the ground water toward the Columbia River; the direction and rate of movement being dependent upon the aquifer characteristics. The need for detailed hydrogeological studies is obvious.

Geological and hydrological studies at Hanford have indicated what aquifers are present and their continuity. Pumping tests have been conducted to determine the field permeability of the sediments, expressed, commonly, as the quantity of water in gallons transmitted daily through each square foot of cross section of the material under a hypothetical slope of water level of one foot per foot. Permeability, together with measurements of water levels in wells to give the actual slope of the water table, gives the average quantity of water moving per square foot of cross section of the aquifer and the approximate direction in which it is moving. Knowing the effective porosity of the material and the quantity of water flowing, the average velocity can be computed. It is therefore possible to estimate the average rate at which radioactive waste would travel if it moved uninhibited with the ground water, about where it would be discharged, and the approximate path it would take to get there.

## OBJECTIVES

It is the purpose of this report (a) to describe the hydrological studies and tests at Hanford which permit the calculation of the hydraulic characteristics of the aquifers present, (b) to determine the general directions and average rates of ground-water flow, (c) to point out important factors which affect the movement of ground water and wastes, (d) to estimate a mean lateral path of potential ground-water contamination from disposal sites to the Columbia River and the "time of travel", and (e) to indicate what is needed in the way of additional geological and hydrological information.

## SUMMARY

The hydraulic characteristics of Hanford aquifers have been measured and estimated by a variety of established field methods. These include the nonequilibrium "type curve" solution, the modified nonequilibrium and Theis recovery formulas, estimations from specific capacity and cyclic fluctuation data, and the results of tracer tests. Mutually consistent results show that the permeability of the glaciofluviatile sediments ranges from about 10,000 gpd/ft$^2$ to more than 60,000 gpd/ft$^2$, and the permeability of the Ringold deposits ranges from about 100 to 600 gpd/ft$^2$. Based on these permeability data, calculations of the average rate of ground-water flow indicate a range from a few inches per day to as much as 160 ft/day.

The hydraulic studies show that wastes which reach the water table beneath disposal sites will potentially move in a general southeastward and eastward direction some 20 miles to the Columbia River. Average rates of ground-water flow indicate that travel along this estimated mean lateral path of ground-water contamination could conceivably be completed in an average time in the order of 180 years. Such factors as heterogeneity and anisotropy of the aquifers, and dispersal of wastes in the ground water,

however, assume great importance in determining the path and ultimately the concentration of radioactive wastes in the water. Consequently, much additional information is needed to determine the quantitative effect of these factors in the field.

## GEOGRAPHY

Hanford Plant lies in a structural and topographic basin west of the southward-flowing Columbia River. The project is bordered on the west by basaltic ridges of the Rattlesnake Hills, Yakima Range, and Umtanum Ridge; on the north by Wahluke Slope which rises gently to the Saddle Mountains; and on the east by a basaltic-lava plateau which is covered near the project by more than 1,000 feet of later sediments now exposed as cliffs on the east bank of the Columbia River.

Land surface ranges in elevation from a maximum of about 3600 feet above mean sea level in the surrounding hills, down to about 700 feet at disposal sites, and to a minimum of about 350 feet where the Columbia River leaves the southern confines of the controlled area.

The drainage basin enclosing Hanford Plant lies west of the project from whence both the infrequent surface runoff and the vastly greater ground-water runoff move across the area and eventually into the Columbia River. The areas primarily involved in the disposal of liquid wastes are located 5 to 15 miles from the Columbia and Yakima Rivers, at locations from which all surface and subsurface runoff is toward the streams. The few wells tapping the ground water for sanitary supplies and downgradient from the disposal sites all are in control of the Atomic Energy Commission and at least five miles from the nearest chemical separations plant, whereas those wells supplying the city of Richland lie several times that distance from the plants.

The Columbia River, which provides cooling and process water for the reactors and separations plants, and sanitary water for plant personnel, varies in flow from about 55,000 second-feet to 100,000 second-feet during the fall and winter months and up to maximum flow rates, recorded during plant operation, of 370,000 to 700,000 second-feet in the spring and early summer.[2]

The Hanford region lies in an intermountain, semi-arid region where the normal total precipitation has averaged 6.29 inches per year for the period 1921-1958. Temperatures range from a mean monthly maximum of 65°F to a mean monthly minimum of 41°F; however, temperatures are recorded from -27°F to 115°F.[3]

## GEOLOGY AND HYDROLOGY

The basaltic lavas forming the bedrock of the Hanford region are folded into a series of anticlines and synclines which trend roughly from northwest to southeast and which generally die out in the Pasco Basin beneath later fill deposits. These later sediments include, in ascending order, (a) the lacustrine Ringold formation, (b) the aeolian Palouse soil, and (c) fluviatile and glaciofluviatile deposits, scabland gravels, and the Touchet sediments. These three are the major geologic units in which the waste products are retained and through which the liquids move.[4]

The lithologic character and water-bearing properties of the several geologic units occurring in the Hanford Area are summarized in Table I.

In general, ground water in the surficial sediments in the Hanford region occurs under water-table conditions although locally artesian conditions caused by cemented deposits and clay lenses exist. Water in the basaltic bedrock at Hanford occurs chiefly under confined conditions, although the water level in wells which penetrate the uppermost part of the basalt generally rises to that of the adjacent unconfined aquifer.

Figure 1 shows contours on the water table and on part of the adjacent piezometric surface as of December 1958. As shown, the natural ground water moves downgradient from the high areas of recharge in the west and southwest toward the Columbia River. The horizontal movement of ground water is somewhat impeded by the basaltic anticline called Gable Mountain

TABLE I

MAJOR GEOLOGIC UNITS IN THE HANFORD REGION AND THEIR WATER-BEARING PROPERTIES

| System | Series | Geologic Unit | Material | Water-Bearing Properties |
|---|---|---|---|---|
| Quaternary | Pleistocene | Fluviatile and glaciofluviatile sediments and the Touchet formation. (0-200 ft thick) | Sands and gravels occurring chiefly as glacial outwash. Unconsolidated, tending toward coarseness and angularity of grains; essentially free of fines. | Where below the water table, such deposits have very high permeability and are capable of storing vast amounts of water. Highest permeability value determined is 66,700 gallons a day per square foot of cross section under a hydraulic gradient of unity. |
| | | Palouse soil (0-40 ft thick) | Wind deposited silt. | Occurs everywhere above the water table. |
| | | Ringold formation (200-1,200 ft thick) | Well-bedded lacustrine silts and sands and local beds of clay and gravel. Poorly sorted, locally semi-consolidated or cemented. Generally divided into the lower "blue clay" portion which contains considerable sand and gravel, the middle conglomerate portion, and the upper silts and fine sand portion. | Has relatively low permeability; values range from 10 to 600 gpd/sq ft. Storage capacity correspondingly low. In very minor part, a few beds of gravel and sand are sufficiently clean that permeability is moderately large; on the other hand, some beds of silty clay or clay are sensibly impermeable under hydraulic gradients of ordinary magnitude. |
| | Miocene and Pliocene | Columbia River basalt series (> 10,000 ft thick) | Basaltic lavas with interbedded sedimentary rocks; considerably deformed. Underlie the unconsolidated sediments. | Rocks are generally dense except for numerous shrinkage cracks, interflow scoria zones, and interbedded sediments. Permeability of rocks essentially nil but transmissibility of a thick section may be considerable. |
| ? | ? | Rocks of unknown age, type, and structure. | Probable metasediments and metavolcanics. | ? |





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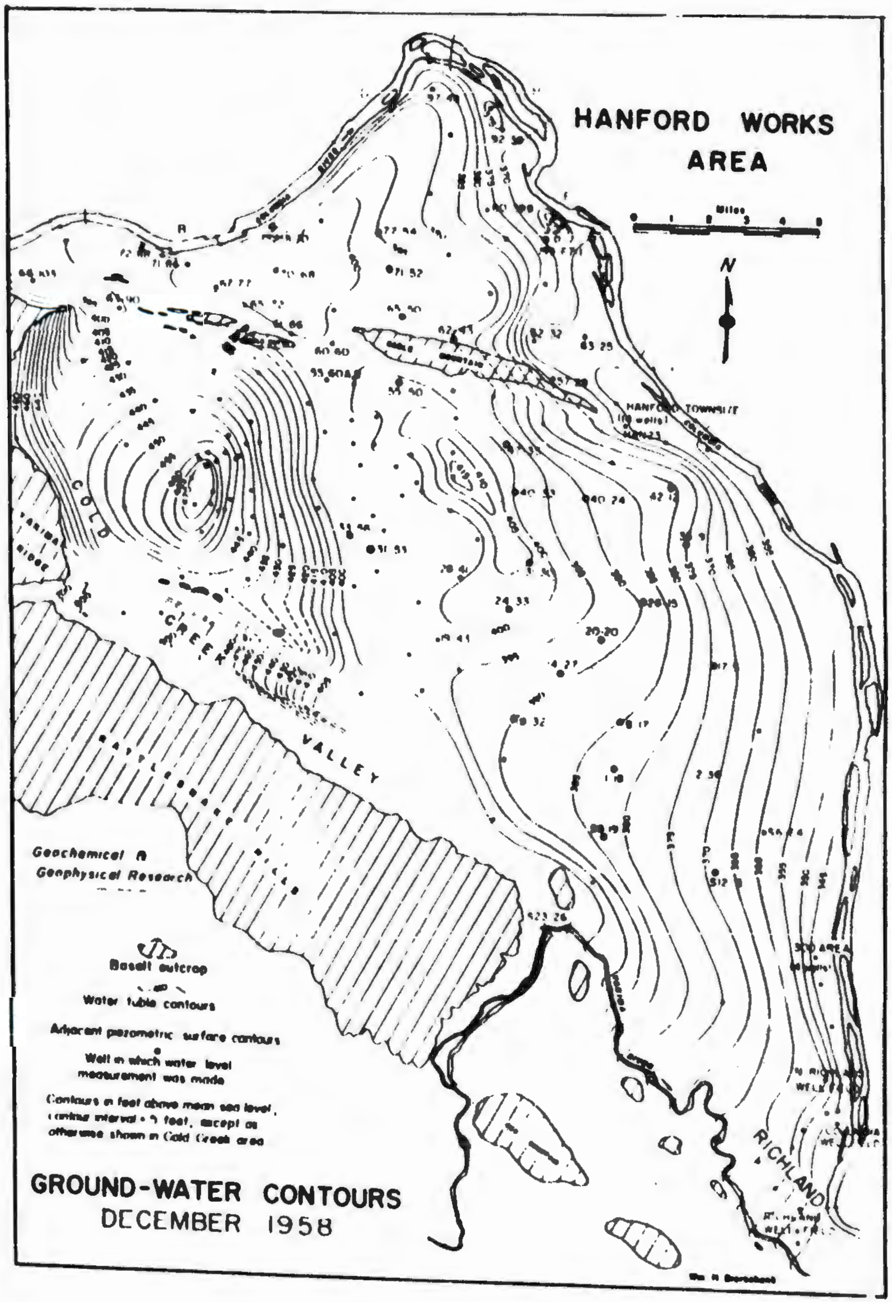


FIGURE 1

Map of Hanford Works Area Showing Ground-Water Contours as of December 1958 and Locations of Wells and Sites Discussed in Text (For convenience, some wells, some well numbers, and the well-number prefix "699-" are omitted.)

and its extensions, and by a buried basalt ridge that parallels Rattlesnake Hills. Adjacent to, and from one to three miles distant from, the Columbia River, the regional body of unconfined water enters a zone in which both direction and rate of movement vary widely, depending on fluctuations of river stage. Discharge of ground water from the area is thus by percolation into the river so long as the river is not at a high stage. The broad, flat mound north of Gable Butte is caused by recharge of adjacent aquifers while the river is at high level. Subsequent to the rapid fall of the river, the ground water escapes only gradually through the sediments. It is thus implied that much of the ground water north of Gable Butte and Gable Mountain is recharged river water, some of which moves eastward and is discharged as springs and seeps to the Columbia River north of the eastern end of Gable Mountain.

Irregularities in the shape and slope of the water table shown in Figure 1 are caused largely by recharge of aquifers by ground-discharged plant effluents. Since 1944 almost 39 billion gallons of waste have been discharged to ground, resulting in the formation of the two distinct ground-water mounds. These mounds have been discussed in detail in an earlier report.[5] It is obvious, however, that the mounds have increased and locally reversed the natural hydraulic gradients and consequently have accelerated the movement of much of the ground water. Irregularities are also caused by differences in the thickness and permeability of the aquifers and by river-level fluctuations.[6]

For further descriptions of Hanford geology and hydrology the reader is referred to earlier reports.[4,7,8]

## MEASUREMENT AND ESTIMATION OF AQUIFER CHARACTERISTICS

### Ground-Water Hydraulics

The movement of ground water in the upper zone of circulation is caused by the differences in hydrostatic head in the different parts of the strata. The relation among the various factors governing the quantity of flow in permeable water-bearing formations can be expressed by a variant of Darcy's law as follows:

$$Q = TIW \qquad (1)$$

where $Q$ = flow in gallons per day through a strip of the aquifer W feet wide with a depth equal to the saturated thickness of the formation,

$T$ = coefficient of transmissibility in gallons per day per foot of width of saturated formation, under a hydraulic gradient of unity,

$I$ = hydraulic gradient in feet per foot, and

$W$ = width of aquifer in feet, perpendicular to the direction of flow.

The value of the coefficient of transmissibility, T, is governed by the thickness and physical properties of the aquifer, particularly the size-variation and packing of the individual particles making up the formation. These physical factors determine the frictional resistances to the flow of ground water. In other words, the coefficient of transmissibility T is equal to the product of the thickness of the aquifer, m, and the field coefficient of permeability, $P_f$; thus $P_f = T/m$. The permeability that represents the proportionality constant between velocity and gradient in the conventional statement of Darcy's law (P) is related to this field coefficient of permeability by:

$$P_f = \frac{1}{m}\int_0^m P\,dm$$

Ground-water movement in the saturated sediments around any system of artificial withdrawal, such as wells, follows Darcy's law, but for quantitative studies certain other formulas provide a better understanding of the relation among the physical characteristics of the water-yielding formation, the time, and the quantity withdrawn.[9]

The withdrawal of water from any aquifer causes the water levels to decline in the vicinity of the point of withdrawal, and the water table around this point assumes an approximate shape of an inverted cone with the apex of the cone at the point of withdrawal. The over-all size, shape, and rate of growth of the cone of depression are dependent on the rate and duration of pumping, the transmissibility and storage coefficients of the aquifer, the increase, if any, in recharge induced by the declining water levels, the natural discharge that is captured, and the hydrogeologic boundaries of the formation. The lowering of the water level is dependent on the variables mentioned above and the distance from the point of withdrawal.

In order to express a general equation for relation among the variables that govern the magnitude of the unwatering effects, certain basic assumptions are made. It is assumed that the formation is constant in thickness, infinite in areal extent, and homogeneous and isotropic (transmits water with equal facility in all directions). It is assumed further that the water may enter the well from the full thickness of the aquifer.

## The Nonequilibrium Formula

The relation among the hydraulic properties of the aquifer, the rate and duration of pumping, and the change in water level caused by the withdrawal of water is expressed in the following form as developed by Theis:[9]

$$s = \frac{114.6\,Q}{T} \int_{\frac{1.87\,r^2 S}{Tt}}^{\infty} \frac{e^{-u}}{u}\,du \qquad (2)$$

where $s$ = drawdown, in feet, at any observation point in the vicinity of the well from which water is withdrawn at a uniform rate,

$Q$ = discharge rate of the well, in gallons per minute (gpm),

T = coefficient of transmissibility, in gpd/ft,

r = the distance in feet, from the point of withdrawal to the point of observation,

S = coefficient of storage, a dimensionless decimal fraction, sensibly equal to the specific yield or effective porosity,

t = time, in days, since the withdrawal of water began.

Methods have been developed for analyzing the nonequilibrium formula and are described by Brown,[10] Wenzel,[11] and others. Theis' "type curve" graphical method of superposition for solution of the equation was described by the writer[12] for three Hanford tests. It will not be repeated here, but an example of such an analysis is given in Appendix I. The results obtained from all four multiple-well tests are summarized in Table II.

## Single Well Pumping Tests

In addition to the four multiple-well tests reported above, 22 single-well tests have been completed at Hanford. All of these were conducted by personnel of the Geochemical and Geophysical Research Operation with equipment provided by the Equipment and Instrumentation Operation. A brief description of this equipment is given in Appendix II. Figure 1 shows the locations of the pumped wells.

Each of the 22 wells was pumped for about 8 hours, and measurement of the drawdown of water level within the well was made periodically. Recovery readings were made for at least 8 hours after pumping stopped. The drawdown-recovery data obtained were analyzed by the modified nonequilibrium method and by the Theis recovery method. The modified nonequilibrium and recovery formulas are presented in Appendix III, together with a discussion of the methods and analyses of test data.

The coefficient of transmissibility was determined at each of the 22 wells by either or both of the single-well methods. The results of the determinations, the estimated effective thickness of the aquifer at each well, and

## TABLE II

## AQUIFER CHARACTERISTICS AS COMPUTED BY NON-EQUILIBRIUM METHOD

| Aquifer Tested | Well No. | Transmissibility (gpd/ft) | Coefficient of Storage | Permeability (gpd/ft$^2$) | Duration of Test (hours) | Distance from Observation Wells to and Nature of, Hydrological Boundary | Number of Observation Wells |
|---|---|---|---|---|---|---|---|
| Glacio-fluviatile | 699-55-50 | 3,000,000 | 0.20 | 66,700 | 48 | None intercepted | 3 |
| Glacio-fluviatile | 699-62-43 | 380,000 | .06 | 12,700 | 169 | 750 ft to discharging boundary; 950 ft to recharging boundary. | 2 |
| Glacio-fluviatile and Ringold | 699-31-53 | 108,000 | .06 | 900 | 8 | 50 ft to discharging boundary | 1 |
| Ringold | 199-K-10 | 34,000 | $7 \times 10^{-5}$ | 400 | 48 | 1,300 ft to recharging boundary. | 2 |

the weighted average coefficient of permeability are given in Table III. Data are included for the four multiple-well tests inasmuch as they also were analyzed by the single-well methods. The tests are tabulated according to the aquifer tested. In several instances it was impossible to differentiate lithologically from drillers' logs between the glaciofluviatile sands and gravels and Ringold sands and gravels. Furthermore, although the Ringold formation is frequently divided into a lower clay zone, a middle conglomerate, and an upper silty zone, marked lithologic changes occur in both vertical and horizontal directions such as to preclude precise definition of zones. The effective aquifer was therefore assumed to include those sediments lying below the water table and above the top of the first relatively impermeable bed - whether silt, clay or basalt. The results of the various pumping tests show that the average field permeability of the several aquifers differ markedly. The one test of the lower Ringold clay gave a low value of 10 gpd/ft$^2$. Excluding the clay zone, the figures obtained for the Ringold formation ranged rather narrowly between the moderate values of about 100 and 600 gpd/ft$^2$. In sharp contrast are the very high permeabilities calculated for the glaciofluviatile sediments - ranging from about 13,000 to 67,000 gpd/ft$^2$. It is apparent that the glaciofluviatile sediments are roughly 100 times more permeable than the Ringold sediments. Intermediate values - 1,000 to 5,000 gpd/ft$^2$ - were obtained from tests wherein the pumped well penetrated sections of both aquifers. In these latter cases the total thickness of the water-bearing material was used, hence the computed permeability is that for the average of the entire section. If the thickness for only the glaciofluviatile material had been used (if known), the computed permeability would approach that for only this more permeable material.

These initially calculated results may require revision on the basis of discoveries resulting from additional testing as the field investigations proceed. The accuracy of the results obtained depends largely on how closely field conditions conform to the basic assumptions of the Theis formula. For example, it is apparent from observation of outcrops and

## TABLE III

### SUMMARY OF PUMPING TEST RESULTS

| Aquifer Tested | Well No. | Transmissibility (gpd/ft) | Estimated Effective Thickness (feet) | Average Field Permeability (gpd/ft$^2$) | Specific Capacity (gpm/ft) | Remarks |
|---|---|---|---|---|---|---|
| Glaciofluviatile sands and gravels | 699-55-50 | 3,000,000 | 45 | 66,700 | 750 | Reference (12); to basalt |
| | -24-33 | 2,900,000 | 45 | 64,500 | 810 | Thickness estimated to top of Ringold |
| | -31-30 | 1,850,000 | 35 | 53,000 | 275 | To top of Ringold |
| | -65-50 | 480,000 | 35 | 13,700 | 37 | To top of Ringold |
| | -62-43 | 380,000 | 30 | 12,700 | 72 | Reference (12), to clay |
| Glaciofluviatile sands and gravels and Ringold sands and gravels, undifferentiated | -42-12 | 645,000 | 130 | 5,000 | 35 | To top of clay |
| | -14-27 | 530,000 | 220 | 2,400 | 165 | Partially penetrating |
| | -77-54 | 318,000 | 65 | 4,900 | 29 | To top of clay |
| | -20-20 | 240,000 | 200 | 1,200 | 19 | Partially penetrating |
| | -40-24 | 210,000 | 150 | 1,400 | 62 | Partially penetrating |
| | -33-56 | 155,000 | 90 | 1,700 | 24 | To top of clay |
| | -31-53 | 108,000 | 120 | 900 | 14 | To top of clay |
| | -26-15 | 67,000 | 45 | 1,500 | 17 | To top of clay |
| | 199-F7-1 | 59,000 | 15 | 3,900 | 14 | To top of clay |
| Ringold sands and gravels with thin silty beds; undifferentiated | 699-2-3 | 92,000 | 160 | 575 | 22 | Partially penetrating |
| | -S8-19 | 80,000 | 160 | 500 | 4 | Partially penetrating |
| | -8-17 | 78,000 | 160 | 490 | 40 | Partially penetrating |
| | -35-9 | 54,000 | 130 | 420 | 3 | Partially penetrating |
| | -71-52 | 30,000 | 50 | 600 | 3 | To top of silt |
| | -17-5 | 8,500 | 45 | 190 | 2 | To top of silt |
| | -8-32 | 4,200 | 40 | 105 | 2 | To basalt? |
| | -S12-3 | 2,100 | 40 | 50 | 2 | To top of clay |
| Ringold conglomerate | -1-18 | 75,000 | 165 | 450 | 43 | To top of clay |
| | 199-K-10 | 34,000 | 85 | 400 | 14 | Reference (12); to clay |
| | 699-47-35 | 3,000 | 20 | 150 | 1 | To basalt |
| Ringold clay | -40-33 | 1,600 | 160 | 10 | <1 | To basalt |

from study of adjacent well logs that the unconsolidated deposits are not homogeneous and that variations in grain size, thickness, and stratification occur within short distances. Furthermore, misleading results may be obtained from partial penetration conditions, under which either the pumped well or an observation well may not be constructed through the full thickness of the formation. The influence of such partial penetration on the results depends on the ratio of vertical to horizontal permeability, the degree of penetration of the pumping and observation wells, and the distance between pumping and observation wells. Generally, values of transmissibility determined by means of the Theis formula from data collected from partially penetrating wells are lower than the values for fully penetrating wells. Methods of correction for partial penetration are available in the literature[11] but these have not always been found to be satisfactory.

## Specific Capacity Tests

The transmissibility of a water-table aquifer may be estimated from the specific capacity of a well; that is, from the yield per foot of drawdown in the well. In many reconnaissance type ground-water investigations, specific capacity data provide the only means of estimating the transmissibility. A coefficient determined in this way is approximate and, generally on the low side because the total drawdown in the well is usually greater than the theoretical formational drawdown. Besides the drawdown in the formation itself, the total drawdown in the well usually includes additional head losses attributable to such extraneous well factors as the size of the well, its position in and percentage penetration of the aquifer, the type and amount of perforation in the casing, and the efficiency of development (see following section, "Step-drawdown tests").

Table IV gives transmissibility values estimated from specific capacity data for 50 wells at Hanford townsite (after which Hanford Plant was named, see Figure 1 for location). Additional specific capacity data are included in Table III. These 50 wells were drilled in the early years of the project, chiefly in 1943, 1944, and 1948, before the beginning of any detailed hydrologic

TABLE IV

ESTIMATED TRANSMISSIBILITY COEFFICIENTS FROM SPECIFIC CAPACITY DATA

| Well Number | Date of Test | Hours of Test | Discharge Q (gpm) | Drawdown s (feet) | Specific Capacity C (gpm/ft) | Estimated Transmissibility (gpd/ft) | Aquifer Thickness (feet) | Average Permeability (gpd/ft²) | Aquifer Gf = glaciofluviatile R = Ringold | Perforation Schedule (length of perforated casing, perforations per foot) |
|---|---|---|---|---|---|---|---|---|---|---|
| [illegible] HAN-1 | [illegible]/43 | ? | 450 | [illegible] | 20 | [illegible] | – | 17,800 | Gf | 5 ft, 12 p/ft |
| -2 | 7/1/43 | ? | 750 | [illegible] | 2,500 | 6,300,000 | 31 | 140,000 | " | 29 ft, 12 p/ft |
| -4 | 6/19/43 | ? | 700 | [illegible] | 1,400 | 2,400,000 | 20 | 120,000 | " | 22 ft, 4 p/ft |
| -5 | 6/22/43 | ? | 750 | 2.7 | 280 | 480,000 | 10 | 48,000 | " | ? |
| -6 | 9/20/43 | 8 | 560 | 2.5 | 225 | 380,000 | 30 | 12,700 | " | 30 ft, 6 p/ft |
| -7 | 9/24/43 | 19 | 1,100 | 2.0 | 550 | 940,000 | 18 | 52,000 | " | 10 ft, 12 p/ft |
| -8 | 10/29/43 | 49 | 1,000 | 9.3 | 110 | 190,000 | 45 | 4,200 | " | 20 ft, 20 p/ft |
| -9 | 12/13/43 | 11 | 1,150 | 14.9 | 77 | 130,000 | 53 | 2,500 | " | 22 ft, 22 p/ft |
| -11 | 7/3/43 | ? | 1,300 | 0.3 | 4,000 | 6,800,000 | 31 | 220,000 | " | 15 ft, screen |
| -15 | 2/6/44 | 27 | 1,170 | 0.4 | 2,900 | 5,000,000 | 45 | 110,000 | " | 26 ft, 28 p/ft |
| -16 | 3/4/44 | 22 | 800 | 0.6 | 1,300 | 2,300,000 | 41 | 56,000 | " | 25 ft, 12 p/ft |
| -17 | 3/21/44 | 29 | 1,125 | 2.3 | 490 | 830,000 | 28 | 30,000 | " | 20 ft, 6 p/ft |
| 23 | 5/2/44 | 34 | 1,125 | 30.2 | 37 | 63,000 | 42 | 1,500 | Gf & R | 5 ft, 10 p/ft |
| -24 | 5/31/44 | 39 | 1,500 | 0.3 | 5,000 | 8,500,000 | 65 | 130,000 | Gf | 40 ft, 10 p/ft |
| 399-[illegible]-2 | 1/30/48 | ? | 750 | [illegible] | 250 | 430,000 | 40 | [illegible] | Gf | " |
| 3-6 | 8/19/43 | 27 | 250 | [illegible] | 500 | 850,000 | [illegible] | [illegible],000 | " | " |
| 3-7 | 1/28/44 | ? | 540 | [illegible] | 900 | 1,500,000 | 45 | 32,000 | " | ? |
| 4-5 | 3/16/58 | 3? | 800 | 64.0 | 13 | 21,000 | 85 | 250 | R | 45 ft, 20 p/ft |
| 1199-15-13C | 3/11/44 | 82 | 2,575 | 30.2 | 85 | 150,000 | 55 | 2,700 | Gf & R | ? |
| 16-13D | 9/11/43 | 48 | 2,266 | 33.3 | 68 | 120,000 | 60 | 2,000 | " | ? |
| 17-13A | 5/12/44 | 58 | 1,350 | 37.7 | 36 | 61,000 | 56 | 1,200 | " | ? |
| 18-13 | 7/29/43 | ? | 920 | 25.6 | 36 | 61,000 | 40 | 1,200 | " | ? |
| 18-13W | 6/9/43 | 13 | 1,350 | 27.4 | 49 | 84,000 | 50 | 1,700 | " | ? |
| 20-13 | 7/23/43 | 7 | 520 | 20.7 | 25 | 43,000 | 30 | 1,100 | " | ? |
| 20-17 | 5/20/43 | ? | 50 | 68.0 | [illegible] | 1,500 | – | ? | Basalt | ? |
| 21-12 | 7/6/44 | 15 | 675 | 13.4 | 50 | 85,000 | 30 | 2,800 | Gf & R | ? |
| 22-11A | 1/11/45 | 105 | 500 | 124.0 | [illegible] | 7,000 | 120 | 60 | R | ? |
| 22-12D | 8/43 | ? | 300 | 94.0 | 3 | 5,[illegible]00 | 105 | 55 | R | ? |
| 1199-32-17 | 5/8/48 | ? | 300 | 4.6 | 65 | 110,000 | 40 | 2,[illegible]00 | Gf & R | ? |
| 33-18B | 2/10/48 | ? | 1,040 | 12.7 | 85 | 140,000 | 40 | 3,500 | " | ? |
| 33-18D | 2/18/48 | ? | 650 | 31.0 | 21 | 38,000 | 19 | [illegible]00 | " | 20 ft, 10 p/ft |
| 33-18E | 4/22/48 | ? | 235 | 4.6 | 51 | 87,000 | 35 | 2,500 | " | 35 ft, 10 p/ft |
| 33-18F | 6/26/48 | ? | 560 | 23.8 | 24 | 40,000 | 35 | 1,100 | " | ? |
| 1199-39-10B | 9/1/44 | ? | 80 | 16.0 | 5 | 8,500 | 40 | 210 | R | ? |
| 39-16A | 5/25/48 | ? | 1,085 | 2.3 | 470 | 800,000 | 35 | 23,000 | Gf | ? |
| 39-16B | 4/1/46 | ? | 660 | 15.0 | 44 | 75,000 | 30 | 2,500 | " | ? |
| 39-16C | 6/26/48 | ? | 1,100 | 3.0 | 370 | 630,000 | 50 | 12,600 | " | ? |
| 39-16D | 4/30/48 | ? | 1,100 | 6.5 | 170 | 290,000 | 25 | 11,600 | " | ? |
| 40-15 | 4/30/58 | ? | 1,000 | 2.0 | 500 | 850,000 | 50 | 17,000 | " | 34 ft, 25 p/ft |
| 40-15A | 4/10/48 | ? | 800 | 17.0 | 47 | 80,000 | 65 | 1,200 | Gf & R | 5 ft |
| 40-15B | 4/30/48 | ? | 1,000 | 6.0 | 165 | 280,000 | 45 | 7,000 | Gf | ? |
| 40-13C | 4/3/44 | 55 | 105 | 26.0 | 4 | 7,000 | 25 | 280 | R | ? |
| 47-18A | 4/14/48 | ? | 230 | 29.0 | 8 | 14,000 | 130 | 110 | R | 38 ft, 2 p/ft |
| 699-S23-26 | 7/12/44 | 30 | 140 | 12.2 | 12 | 20,000 | 30 | 700 | Gf & R | ? |
| 55-60A | 12/30/43 | 118 | 565 | 2.0 | 280 | 480,000 | 55 | 8,700 | Gf | 40 ft, 6 p/ft |
| 55-60B | 8/16/44 | 26 | 1,280 | 0.9 | 1,400 | 2,400,000 | 110 | 22,000 | Gf | 55 ft, 22 p/ft |
| 57-29 | 2/17/50 | 1 | 500 | 2.0 | 250 | 430,000 | 35 | 12,300 | Gf | ? |
| 63-25 | 8/49 | ? | 370 | 2.5 | 150 | 250,000 | 75 | 3,300 | Gf & R | 30 ft, 4 p/ft |
| 66-103 | 1/26/44 | 24 | 540 | 0.6 | 900 | 1,500,000 | 55 | 27,000 | Gf | 50 ft, 8 p/ft |
| 89-39B | 2/10/44 | 24 | 167 | 6.8 | 28 | 48,000 | 24 | 2,000 | Gf & R | 24 ft, 12 p/ft |

investigations. The only quantitative information available, therefore, was that included in drillers' well completion reports. The values for T were estimated from equation (3). Equation (3) and the relationship between specific capacity and estimated transmissibility are discussed in Appendix IV.

$$T = \frac{Q}{s}(1700) \qquad (3)$$

where T is the coefficient of transmissibility (gpd/ft), Q is the discharge (gpm), and s is the total drawdown (feet) in the pumped well.

Most of the wells listed in Table IV are situated in five general areas: Hanford townsite, 300 Area, North Richland well field, Columbia well field, and Richland well field. The remaining seven wells are scattered throughout the project (see Figure 1 for location of areas and well sites).

The specific capacities for the 14 wells at Hanford townsite (699-HAN-1 to 24) range from 30 to 5,000 gpm/ft and the corresponding estimates for transmissibility range from about 50,000 to 8,500,000 gpd/ft. These wells derive their water from glaciofluviatile deposits of sand, gravel, and boulders which overlie Ringold clays or silts. The relatively small values reflect the thinness of the aquifer at specific sites and the extremely high values may reflect river recharge. The average value for transmissibility of 2,300,000 gpd/ft does not appear unreasonable, and estimating the aquifer to be about 35 feet thick gives a field coefficient of permeability of 65,000 gpd/ft$^2$. This is in close agreement with the values calculated for similar glaciofluviatile sediments elsewhere (Table III). Piper[13] assumed an over-all average permeability of 35,000 gpd/ft$^2$ based on specific capacity reports for seven wells.

Three wells in the 300 Area (399-3-2,6,7) were tested and the specific capacity data indicate a range in transmissibility from about 400,000 to 1,500,000 gpd/ft. The aquifer thickness varies with changing river level, but assuming the thickness of the glaciofluviatile sands and gravels to be 40-45 feet, the permeability ranges from about 10,000 to 30,000 gpd/ft$^2$.

One recent test (399-4-5) of the underlying Ringold formation indicates a transmissibility of 21,000 gpd/ft and a permeability of 250 gpd/$ft^2$.

Specific capacity data for 7 of the 10 wells in the Richland well field (1199-15- to 22-) indicate an average transmissibility of about 85,000 gpd/ft. These wells tap glaciofluviatile and Ringold sediments so that the over-all average permeability approaches 2,000 gpd/$ft^2$. Two of the wells tap only the Ringold sediments, and for these the estimated permeability was about 60 gpd/$ft^2$. Piper(13) reported that the permeable sands and gravels occurred as channel-fill deposits between older terrace deposits of compact clay or siltstone. He estimated an average permeability of about 8,000 gpd/$ft^2$ from data from four tests.

The Columbia well field (1199-32- to 33-) is adjacent to the west bank of the Columbia River. Data from tests of five wells tapping the glaciofluviatile and Ringold sediments above the Ringold clays indicate average permeabilities in the order of 1,000 to 4,000 gpd/$ft^2$.

Data from the wells in or near the North Richland well field (1199-39-to 40-) indicate that the glaciofluviatile deposits have an estimated permeability of about 7,000 to 23,000 gpd/$ft^2$. The one well withdrawing water from both glacial and Ringold deposits gave an average permeability of about 1,000 gpd/$ft^2$, and the three wells tapping Ringold deposits gave permeabilities ranging from about 100 to 300 gpd/$ft^2$.

In summary, the specific capacity data for the 43 wells situated in the five areas along the west edge of the Columbia River, as well as for the seven wells scattered elsewhere on the Hanford project, permitted estimates to be made of the transmissibility of the deposits tapped. Admittedly most of these estimates are undoubtedly on the low side; nevertheless the values subsequently obtained for average permeability are generally in the same order of magnitude as those obtained from controlled pumping tests for similar aquifer materials

That is, in general,

(a) Glaciofluviatile deposits . . . . . . . . . . $P > 10{,}000$ gpd/ft$^2$.
(b) Glacial and Ringold deposits . . . . . . . $P = 1{,}000 - 3{,}000$ gpd/ft$^2$,
(c) Ringold deposits . . . . . . . . . . . . . $P = 100 - 300$ gpd/ft$^2$.

Step-Drawdown Tests

The drawdown in a pumped well has two components; the first, arising from the "resistance" of the formation, is proportional to the discharge; and the second, termed "well loss" and representing the loss of head that accompanies the flow through the perforations and upward inside the casing to the pump intake, is proportional approximately to the square of the discharge. The resistance of an extensive aquifer increases with time as the ever-widening area of influence of the well expands.[14] Consequently, the specific capacity of the well decreases both with time and with discharge.

Step-drawdown tests were conducted on 14 wells. Inasmuch as the step-drawdown analysis yields information primarily concerning the performance of the wells rather than the aquifers penetrated, the tests were conducted principally in an attempt to evaluate the worth of the wells for radiologic monitoring purposes. In order for a water sample to be reasonably representative of the ground waters outside the well, free communication through casing openings is necessary. If the openings are partially plugged, corroded, encrusted, or otherwise inadequate, radiochemical analyses of the well water will be of questionable value in studies of ground-water contamination. The step-drawdown tests were employed to determine the relative "efficiency" of the wells. A discussion of theory and an analysis of test data are given in Appendix V.

Data obtained from the multiple-step drawdown tests are tabulated in Table V. In Figure 2 are shown "well-efficiency" curves for the wells at various hypothetical pumping rates. For this plotting "well-efficiency" is defined as the ratio of (a) the theoretical drawdown computed by assuming no turbulence is present, or approximately $BQ$, where $B$ is the formation-loss constant and $Q$ is the discharge, to (b) the drawdown in the well, $s_w$. These curves show a rapid drop in efficiency as discharge is increased.

TABLE V

MULTIPLE-STEP DRAWDOWN TESTS ON PERFORATED WELLS

| Well Number | Date of Perforating | Perforation Schedule (length of casing, perforations per round, and rounds per foot) | Date of Test | Results of Test $s_w = BQ + CQ^2$: Formation-loss Constant, B | Results of Test $s_w = BQ + CQ^2$: Well-loss Constant, C |
|---|---|---|---|---|---|
| 699-S8-19 | Aug 1950 | 15 ft, 4 p/rd, 1 rd/ft | June 1958 | 0.06 | [illegible] |
| -2-3 | May 1950 | 15 ft, 4 p/rd, 1 rd/ft | Oct 1958 | 0.025 | 0.00016 |
| -33-56(b) | Screened | 124 ft, No. 20-slot well screen, not developed | Nov 1958 | 0.009 | 0.00018 |
| -8-17 | May 1950 | 15 ft, 4 p/rd, 1 rd/ft | June 1958 | 0.0125 | 0.0001[illegible] |
| -26-15 | Jan 1958 | 95 ft, 2 p/rd, 1 rd/ft | July 1958 | 0.044 | 0.00009 |
| -S8-19(a) | June 1958 | 8 ft, 4 p/rd, 4 rd/ft | June 1958 | 0.073 | 0.000052 |
| -1-18 | Jan 1958 | 100 ft, 1 p/rd, 1 rd/ft | | | |
| | May 1958 | 8 ft, 4 p/rd, 4 rd/ft | May 1958 | 0.0122 | 0.00048 |
| -65-50 | Aug 1955 | 55 ft, 4 p/rd, 1 rd/ft | May 1957 | 0.0115 | 0.000041 |
| 199-K-10(b) | Aug. 1952 | 17 ft, 4 p/rd, 1 rd/ft | Sept 1952 | 0.0544 | 0.000029 |
| 699-31-53(b) | Screened | 117 ft, No. 35-slot well screen, developed | April 1959 | 0.0134 | 0.000012 |
| -14-27 | April 1956 | 30 ft, 2 p/rd, 1 rd/ft | May 1958 | 0.00085 | 0.000012 |
| -26-15(a) | July 1958 | 12 ft, 4 p/rd, 2 rd/ft | July 1958 | 0.037 | 0.000012 |
| -31-30 | Feb. 1956 | 67 ft, 2 p/rd, 1 rd/ft | July 1957 | 0.00115 | 0.0000085 |
| -55-50 #1 | June 1956 | 45 ft, 6 p/rd, 2 rd/ft | Feb 1957 | 0.00082 | 0.000004 |
| -24-33 | Aug. 1948 | 20 ft, 5 p/rd, 1 rd/ft | | | |
| | Sept. 1956 | 15 ft, 2 p/rd, 1 rd/ft | Aug 1957 | 0.00022 | 0.0000032 |
| -55-50 #2(b) | Screened | 45 ft, No. 60-slot well screen, developed | Sept 1956 | 0.00122 | 0.00000017 |

(a) Test rerun after jet perforating.

(b) 12-inch diameter well, all others are 8-inch diameter

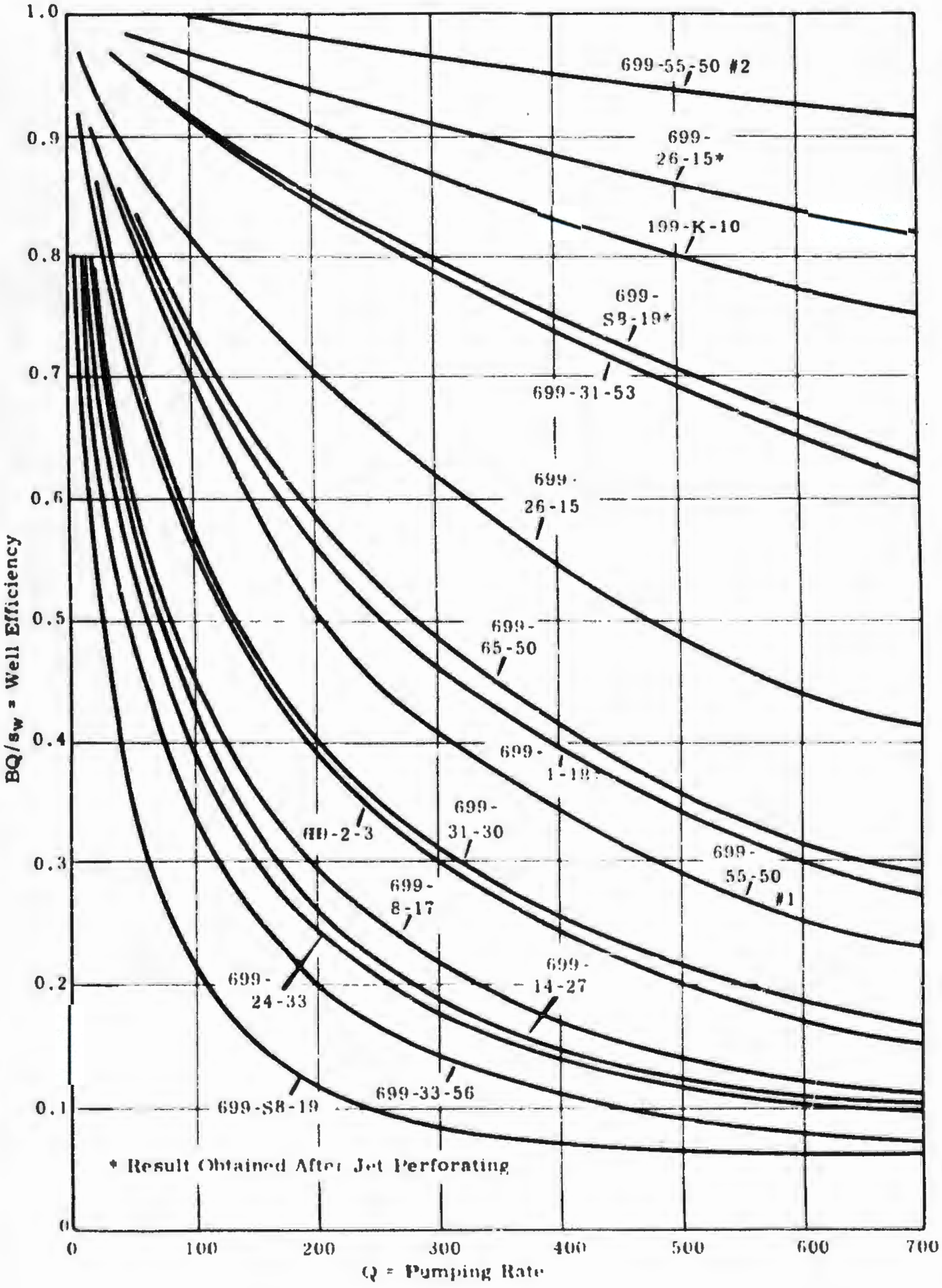


FIGURE 2

Graph Showing 1-Hour Well Efficiency

A brief study of Table V and Figure 2 reveals that those wells with the greatest declines in efficiency were either drilled 8 to 10 years before the test, or else originally were meagerly perforated considering the formation they tapped. For example, wells 699-S8-19, -2-3, and -8-17 were completed in 1950, each having a total of only 60 perforations. At a pumping rate of 100 gpm, the respective well efficiencies are 21 per cent, 56 per cent, and about 45 per cent. Wells 699-24-33, -14-27, and -31-30 each tap an extremely permeable aquifer (see Table III) so that despite their relatively small well-loss constants, C, their efficiency is low because of correspondingly very small formation-loss constants, B. At 100 gpm the respective efficiencies are about 39 per cent, 43 per cent, and 58 per cent.

Two wells were tested twice. Well 699-S8-19 was tested "as is" on June 7, 1958 and the total drawdown of the pumped well was calculated as: $s_w = 0.06\ Q + 0.0022\ Q^2$. The well was then reperforated by the shaped-charge jet technique,[15] and another step-drawdown test was run on June 9, 1958. The total drawdown was then defined as: $s_w = 0.073 + 0.000062\ Q^2$, indicating a much more efficient well. From Figure 2 it is seen that originally at 100 gpm the well efficiency was only about 21 per cent, but after jet perforation it was about 92 per cent. Likewise for well 699-26-15, the efficiency at 100 gpm was increased from an original 82 per cent to about 97 per cent after jet perforating. The value of adequate perforations, both in number and distribution, is clearly demonstrated.

It is evident from the curves of Figure 2 that many of the wells tested are "inefficient" in regard to the higher pumping rates. Whether or not these same wells are valueless for monitoring purpose is uncertain. If the graph of well efficiency for well 699-55-50 #1 is arbitrarily chosen to represent acceptable standards, it is seen that the six curves to the left would indicate wells whose free interconnection with the aquifers penetrated is suspect. Well 699-55-50 #1 was selected because it is a fully penetrating well with adequate perforations.

## Tracer Tests

Field determinations of permeability were made at Hanford by the velocity method in which one well was used for the injection of dye and wells down-gradient were used as observation wells. The time rate of travel of the injected substance through the water-bearing material was thus determined.

Inasmuch as the velocity of ground water is directly proportional to the permeability of the material through which it moves and to the hydraulic gradient, the field coefficient of permeability may be computed from

$$P_f = \frac{7.48\, pv}{I} \qquad (4)$$

where $P_f$ is the field coefficient of permeability (gpd/ft$^2$), p is the effective porosity, v is the ground-water velocity (ft/day) and I is the hydraulic gradient (ft/ft).

### Characteristics of the Ideal Tracer

As reported by Kaufman and Orlob,[16] a satisfactory tracer (a) should be susceptible to quantitative determination in very low concentration, (b) should be entirely absent from the injected water or present only at low concentrations in the displaced water, (c) must not react with the injected water or displaced waters to form a precipitate, (d) must not be absorbed by the porous medium, (e) must be cheap and readily available, and (f) must not undergo such physical or chemical change during passage through the ground as to impair the degree of precision of detection.

As the result of laboratory and field studies, Kaufman and Orlob[16] concluded that sodium fluorescein is probably the most satisfactory of the dyes available, but that its use should be limited to granular media free of organic material and of relatively high permeability (such as at Hanford). Consequently, commercial fluorescein, or uranin, was used for tracing ground water at three sites on the Hanford project.

## Results of Field Tests

In April 1954, a tracer test was conducted at the 699-62-43 site using wells that are 50 feet apart. As reported by Honstead et al,[17] the rate of ground-water movement was measured at 170 ft/day. For an effective porosity of 0.06[12] and a hydraulic gradient of 0.0026 ft/ft, the permeability is calculated to be 29,000 gpd/ft$^2$. In comparison, the average permeability determined from the multiple-well pumping test at this site was 12,700 gpd/ft$^2$.[12]

On July 27, 1956, a solution of 100 pounds of fluorescein in 100 gallons of water was bled into well 699-28-41. Water samples were taken several times a week from well 699-19-43, located about 8,800 feet to the south-southwest. Concentration of fluorescein in samples was determined by the General Chemical Analysis Operation at Hanford as follows: (a) the samples were concentrated from 1 liter to 100 ml, (b) the fluorescein was extracted with isoamyl alcohol, and (c) the fluorescence of the alcohol phase was measured with the JACO 200 Fluorimeter. The method can detect one-tenth of one part per billion, or a reduction in concentration to about $10^{-9}$ of the initial concentration. Figure 3 shows the curves of fluorescein concentration as the dye passed the observation well. At this time the eastern ground-water mound had not attained the size or shape as shown in Figure 1, and a gradient existed from 699-28-41 to 699-19-43. The buildup of the mound, however, resulted in anomalous sample results from three other observation wells, and these were not further evaluated.

On September 9, 1957, a similar tracer test was initiated, using well 699-24-33 as the injection well, well 699-14-27, located about 11,500 feet south-southeast as one observation well, and well 699-20-20, located about 13,200 feet east-southeast, as a second observation well. Figure 3 shows the time-concentration curves of fluorescein for the two observation wells.

Table VI summarizes the data collected and used for computing field permeability between the test wells. An effective porosity of 10 per cent is assumed for computation of equation (4).

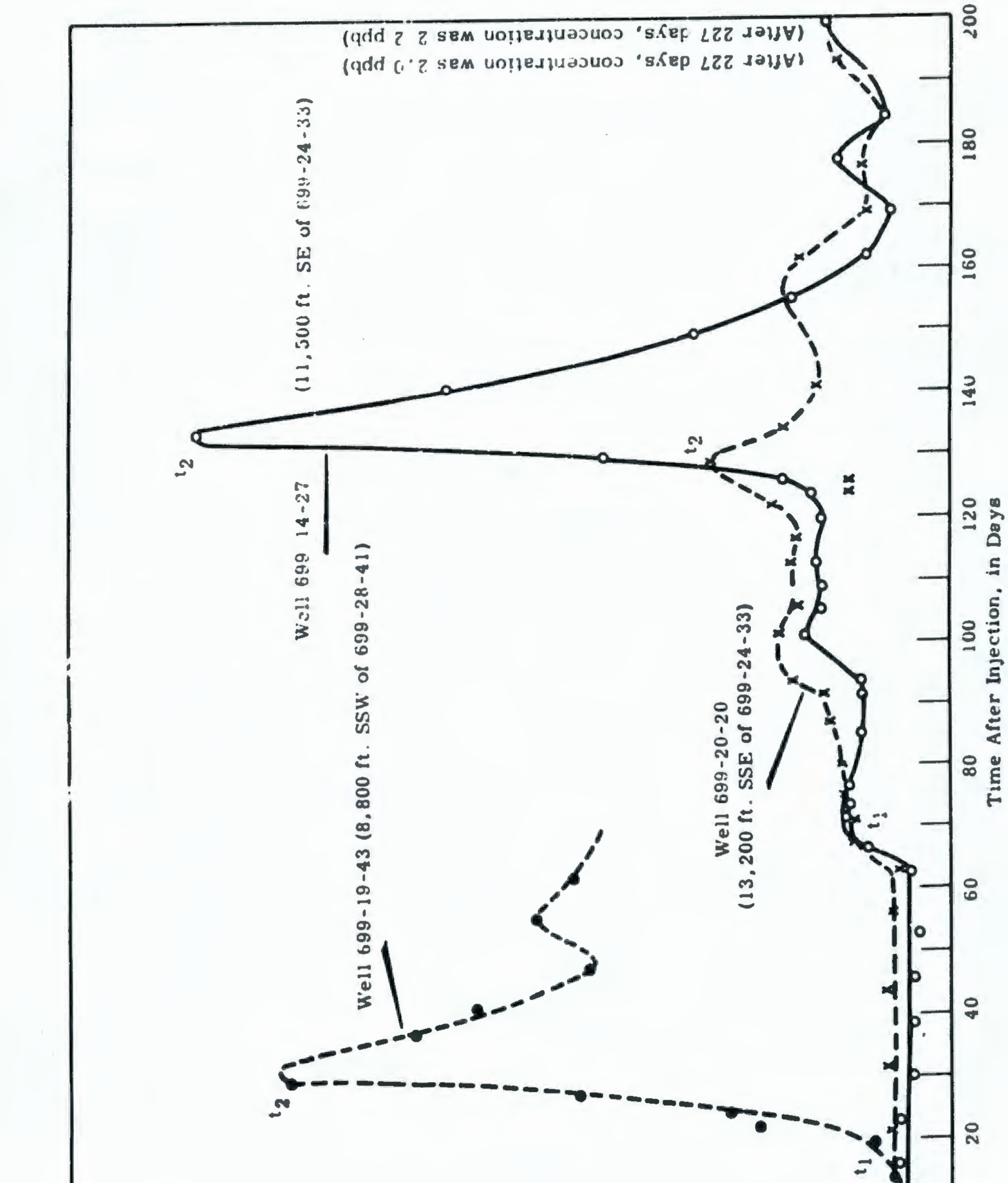


FIGURE 3

Time-Concentration of Fluorescein in Observation Wells

**TABLE VI**

**TRACER VELOCITY DATA**

| Observation Well Number | Distance from Point of Injection (feet) | Travel Time in Days: First Detection $t_1$ | Travel Time in Days: Modal Value $t_2$ | Velocity in ft/day: First Detection $v_1$ | Velocity in ft/day: Modal Value $v_2$ | Permeability in gpd/ft$^2$ Based on: $v_1$ | Permeability in gpd/ft$^2$ Based on: $v_2$ |
|---|---|---|---|---|---|---|---|
| 699-19-43 | 8,800 | 20 | 31 | 440 | 280 | 165,000 | 105,000 |
| -14-27 | 11,500 | 67 | 134 | 170 | 85 | 125,000 | 63,000 |
| -20-20 | 13,200 | 67 | 129 | 195 | 100 | 145,000 | 75,000 |

The time of first detection, $t_1$, is an event of primary importance because it signifies that the dye has moved from the point of injection to the point of observation. The velocity determined by $t_1$ indicates a ground-water component moving at least this rapidly in this direction. In the absence of better information, it is assumed that this will probably approach the maximum velocity of the ground water (discounting any adsorption effects) in the particular section tested, and therefore gives results for permeability that are too high for the average of the entire section tested. The curves of Figure 3 show that after the first arrival, greater concentrations of the dye (or contaminant) traveling with the ground water may be expected. Permeability values based on the time of arrival of the modal value or maximum detected concentration ($t_2$) appear more typical of the average calculated from pumping tests. Comparing the high permeability results determined by the velocity method to those determined by pumping tests gives some basis for judging aquifer variability. In other words, it appears that the permeability of some beds inthe glaciofluviatile sediments tested may be as much as three times the average value computed by aquifer tests, and that the rate of travel of the fastest thread of water is likewise as much as three times the average velocity.

The after-peaks shown in the curves of Figure 3 suggest that the tracer arrived at the observation wells by more than one route or perhaps through different sedimentary beds. Furthermore, the 30° angle subtended from the point of injection (699-24-33) to the points of detection (699-14-27 and -20-20) suggests the degree of transverse dispersal of the tracer that may have occurred.

## Cyclic Fluctuations of Water Level

The response of parts of Hanford aquifers to the passage of yearly flood crests in the hydraulically connected Columbia River may be used as a basis for estimating aquifer transmissibility. As the stage of the river rises, the head upon the subaqueous outcrop of the aquifer increases and thereby either increases the rate of inflow to the aquifer or reduces the rate of outflow therefrom. The increase in recharge or decrease in discharge results in a general recovery of water level in the aquifer. On the subsequent falling stage this pattern is reversed. The water levels in 15 wells at Hanford fluctuate in response to yearly sinusoidal changes in river stage.[6]

A brief discussion of the methods used for estimating transmissibility from cyclic fluctuation data and an analysis of field data are given in Appendix V. Table VII summarizes the tentative results obtained.

TABLE VII

SUMMARY OF RESULTS FROM CYCLIC FLUCTUATION DATA

| Aquifer | Well No. | Transmissibility (gpd/ft) | Estimated Effective Thickness (feet) | Avg. Field Permeability (gpd/ft$^2$) | Remarks |
|---|---|---|---|---|---|
| Glacio-fluviatile | 699-60-60 | | | | S assumed to be 0.06 based on result of multiple-well test at site 62-43 north of Gable Mt. |
| | -61-66 | | | | |
| | -65-72 | 2,300,000 | 40 | 57,000 | |
| | -63-90 | | | | |
| | -66-103 | | | | |
| | -57-29 | 610,000 | 35 | 17,000 | |
| | -62-32 | 790,000 | 35 | 23,000 | |
| Glacio-fluviatile and Ringold | -63-25 | 130,000 | 75 | 1,700 | S assumed to be 0.10 |
| | -67-77 | 190,000 | 120 | 1,600 | |
| | -70-68 | 240,000 | 120 | 2,000 | |
| | -HAN-23 | 260,000 | 42 | 6,200 | |
| Ringold | -71-84 | 15,000 | 100 | 150 | |
| | -72-88 | 51,000 | 100 | 510 | |
| | -92-38 | 32,000 | 100 | 320 | |

The above indicated values of the coefficients of transmissibility and permeability are considered tentative estimates. These data serve merely to demonstrate the applicability, usefulness, and limitations of the method described for analyzing cyclic fluctuations of ground-water level. However, the results obtained are in the same order of magnitude as those reported previously. Thus, the permeability of the glaciofluviatile sediments ranges from 17,000 to 57,000 gpd/$ft^2$; the permeability of the Ringold sediments ranges from about 100 to 500 gpd/$ft^2$; and intermediate values of about 1,000 to 6,000 gpd/$ft^2$ were obtained when considering a section of both glaciofluviatile and Ringold sediments.

## Other Quantitative Studies

### Gradient Method

In the vicinity of the western ground-water mound the Ringold formation presumably is quite homogeneous, as evidenced by the roughly conical shape of the ground-water contours (Figure 1). An approximate value for transmissibility may be obtained by considering the flow through one or more of the closed contours of the mound. Thus, according to a modified form of Darcy's law, the gradient formula for transmissibility(11) is

$$T = \frac{Q}{IL} \qquad (5)$$

where T is the transmissibility (gpd/ft), Q is the average discharge into the mound (gpd), I is the average hydraulic gradient around the contour (feet/mile), and L is the length around the contour (miles).

Considering the 450-foot contour onthe western mound (see Figure 1), a weighted average gradient of 15.3 feet/mile was determined by averaging the gradients for several segments around the contour. The length of the contour is about 10.3 miles and the average input to the mound in the period December 1944-September 1958 was roughly 4,200,000 gpd. From equation (5),

$$T = \frac{4,200,000}{(15.3)\ (10.3)} = 27,000 \text{ gpd/ft}$$

Newcombe and Strand(18) calculated the transmissibility to be about 25,000 gpd/ft based on similar data for the period 1944-1953. A moot question was raised by them involving the thickness which should be used for the aquifer. It was reasoned that under normal conditions the aquifer consisted of the saturated part of the conglomerate member of the Ringold formation and that undoubtedly the entire saturated part of the conglomerate member is used to some extent in the outward transfer of the water from the recharge mound. But the nonisotropic and layered character of the material must preclude an ideal flow pattern vertically across the stratification downward and laterally outward from beneath the mound. The actual shape of the flow net was consequently assumed to be something between the two idealized concepts of: (a) downward and laterally outward by a uniform migration in a symmetrical flow net, as is common to isotropic materials, and (b) an entirely lateral slippage off the mound without deep circulation. Newcombe and Strand subsequently estimated a saturated thickness of 240 feet, which gave an average permeability of 105 gpd/ft$^2$. Present knowledge of the area shows that the effective saturated thickness ranges from 100 feet in the northeast to 220 feet in the southwest. In any event, the permeability of the Ringold formation here is probably within the range of 100 to 300 gpd/ft$^2$.

Porosity Determinations

The median specific yield or the available porosity (volume saturated less the specific retention) of the sediments being used for storage of the recharged waters of the ground-water mounds was estimated by Parker and Piper(19) to be about 4.8 per cent in the west and about 6.5 per cent in the east. Newcombe and Strand(18) obtained approximate values for available porosity by dividing the total amount of water delivered to the respective plants during the period November 1948-May 1953 into the volume of the mound's growth for the same period. Those quantities gave a value of about 11 per cent for both the western and eastern areas. In a similar manner, the writer(6) calculated the wetted porosity to be about 6.0 per cent for the saturated sediments of the western mound and about 7.2 per cent for those of the eastern mound, and averaging about 6.4 per cent. These figures were based on the following data:

(a) Total volume of sediments affected by rise of water levels = 62 billion cubic feet

(b) Total volume of effluents discharged to ground December 1955-June 1957 = 4 billion cubic feet.

It was assumed that all effluents were still in the zone of saturation and that none had been discharged into the Columbia River.

Based on the various direct and indirect methods of calculation, the effective porosity of Hanford aquifers averages about nine per cent; a rounded-off value of ten per cent is used for most subsequent calculations.

## Applicability of Methods

The methods discussed for measuring and estimating aquifer characteristics have been used successfully by hydrologists for many years. It must be recognized, however, that the numerical results obtained by substituting aquifer test data in an appropriate mathematical model indicate the transmissibility and storage coefficients for an ideal aquifer. If enough data are always available it will be found that no ideal aquifer could reproduce the data obtained, for example, in an actual pumping test. The dispersion of the data actually is a measure of how far the aquifer departs from the ideal. Recognized departures from the ideal, however, do not constitute grounds for abandoning, or rarely using, available analytical equations.

The various methods described were used to determine the permeability of Hanford aquifers. A summary of the results obtained by these methods is given in Table VIII.

### TABLE VIII

### SUMMARY OF AQUIFER PERMEABILITY

| | Avg. Field Permeability (gpd/ft$^2$) Calculated from: | | | | |
|---|---|---|---|---|---|
| Aquifer Tested | Pumping Tests | Specific Capacity Tests | Tracer Tests | Cyclic Fluctuations | Gradient Method |
| Glacio-fluviatile | 13,000-67,000 | 10,000-65,000 | > 63,000 | 17,000-57,000 | --- |
| Glacio-fluviatile and Ringold | 900-5,000 | 1,000-4,000 | --- | 1,600-6,000 | --- |
| Ringold (excluding clay zone) | 50-600 | 60-300 | --- | 150-500 | 100-300 |

The extraordinary variability in the coefficients of permeability and transmissibility point out that one quantitative test could not satisfy the demand for a quantitative study of Hanford aquifers. Each test is merely a guidepost or segment of knowledge. Fortunately, at Hanford, individual tests have been supported by a variety of additional tests such that the aquifer characteristics can be stated with considerable confidence. As stated previously, knowledge of the permeability and the effective porosity, together with measurements giving the actual slope of the water table, determines the quantity of ground-water flow and its approximate direction and permits the computation of average velocity. Such determinations are given in Figure 4and are discussed in more detail in following sections. The areal distribution of glaciofluviatile sediments shown in Figure 4 is partly based on the foregoing hydrogeologic data, and partly on an unpublished contour map of the top of the Ringold formation. *

## MOVEMENT OF GROUND WATER AND CONTAMINATION

### Directions of Movement

In general each water particle in the zone of saturation is moving from some point in an intake area where it first reached the water table toward some point where water is being discharged through springs or seeps, or by evaporation or adsorption by the roots of plants.[20] The energy that keeps the water in motion against the internal friction created by its own viscosity is provided by the difference in head between the place of intake and the place of discharge. The path followed by each thread of water leads in varying directions, and at times it may have an upward trend, but the place of discharge is at a lower level than the place of recharge, and thus potential energy is lost. The difference in head is distributed throughout each thread of water as a hydraulic gradient, continuously but not at a constant rate, all the way from intake to exit. In the absence of more precise data, it is assumed that ground water always moves in the direction of the hydraulic

* Personal communication from R. E. Brown.

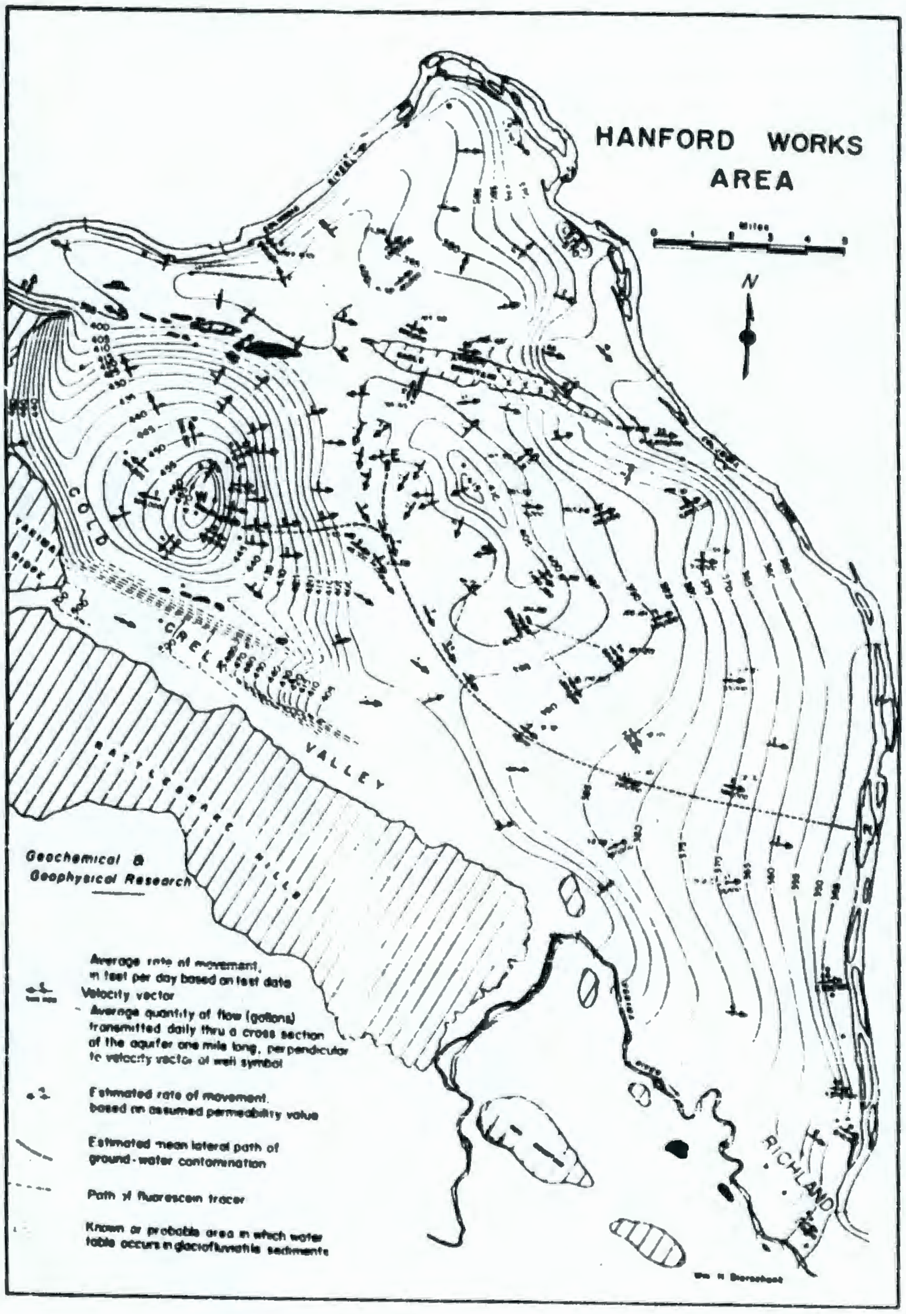


FIGURE 4

Map of Hanford Works Area Showing General Direction and Average Rate of Ground Water Movement gradient. Therefore, the best means of determining the direction of movement is by drawing lines perpendicular to ground-water contours, from high to low head (see velocity vectors, Figure 4). Strictly, however, even a perfect contour map of the water table would show only the horizontal direction of movement of the ground water at the water table. The hydraulic gradients are three-dimensional, however, and the water moves not only along the water table but also to depths below the water table and generally upward again to the water table at some other place.

The ground-water contour maps (Figures 1 and 4) are based on the measured altitude of the water surface in a pattern of wells (at present numbering 130), the contour lines representing lines of equal altitude on the water table or piezometric surface expressed in feet above mean sea level. The greatest number of these wells monitor the two ground-water mounds permitting rather accurate contouring in these locations. The relatively few data available for those areas lying north of Gable Mountain and immediately northeast of Rattlesnake Hills require liberal interpretation and generalization, and there may be substantial deviation, at least in detail, from the shape and position of the contours as shown.

As Figure 4 indicates, the present pattern of ground-water movement underlying Hanford Plant has changed fundamentally during the 14 years of plant operation, owing to concurrent changes in water-table form.[5] In brief, the zone saturated by infiltered waste effluents creates a ground-water divide, roughly concave to the south and enclosing disposal sites on the west, north, and east. From the northern or outer flank of this divide, the artificially recharged water largely moves radially northwestward and northeastward. From the southern or inner flank of the divide, the infiltered wastes converge and move generally southeastward and then more eastward in a relatively narrow band.

Owing to these artificial elements in the pattern of movement, ground water which enters the area naturally, along Cold Creek valley at the base of Yakima Ridge and Rattlesnake Hills, presumably now can pass to the Columbia River only upstream from the western ground-water mound and downstream from a point about 10 miles north of the Yakima River. It is now excluded from the intervening reach of the river.

The directions of movement in the regional body of unconfined water, here described, are those which would be taken currently by any radioactive waste products infiltrating to the water table from an overlying disposal site.

## Limiting Cases

In order to predict the movement of any radiocontaminants, several important factors must be considered. These include the movement of water, the dispersal of matter in water, and the adsorption of substances by natural materials in contact with water.

The movement of ground water has been studied for many years, and mathematical models have been worked out for many of the limiting cases. Low-level radioactive wastes that enter the water table beneath disposal sites W and E shown in Figure 4 will move from these places of injection under the increased head imposed by the injection and also under the hydraulic gradient under which the natural ground water is moving. As pointed out by Theis,[21] in a homogeneous and isotropic formation the mathematics of ground-water flow is analogous to that of the conduction of heat or electricity, with the concept of hydraulic head taking the place of that of temperature or voltage, and permeability that of thermal or electrical conductivity. Hence the shape and size of a volume of waste in the ideal formation can be computed.

If wastes of the same viscosity and density are injected at a fixed rate into an ideal formation, the volume of aquifer occupied by wastes will have a lateral short axis normal to the gradient equal to 2a. It will reach a point upgradient a distance, b, at which the imposed hydraulic gradient will equal the oppositely directed natural gradient. Its course downgradient will be governed by the imposed and natural gradients and it will reach a distance, c, in the time, t. These distances are given by the following formulas:[21]

$$a = \frac{Q}{2\,PIm} \qquad (6)$$

$$b = \frac{Q}{2\pi\,PIm} \qquad (7)$$

$$t = 7.48\left(\frac{p}{PI}\right)\left[c - b\ln\left(1 + \frac{c}{b}\right)\right] \qquad (8)$$

where

$Q$ = the input into the aquifer in gallons/day (gpd),
$P$ = permeability of the aquifer (gpd/$ft^2$),
$m$ = thickness of aquifer (ft),
$p$ = porosity of the aquifer,
$I$ = natural hydraulic gradient (ft/ft).

Considering a waste discharge of $Q$ = 1,000,000 gpd into the glaciofluviatile sediments where $P$ = 60,000 gpd/$ft^2$, $m$ = 40 ft, $p$ = 0.10, and $I = 2 \times 10^{-4}$ (roughly 1 ft/mile), the dimensions would be: $a \approx$ 1,000 ft, $b \approx$ 330 ft, and $c \approx$ 2,200 ft after 100 days of injection and about 18,000 ft in 1,000 days. For the Ringold formation in which $P$ = 500 gpd/$ft^2$, $m$ = 200 ft, $I = 10^{-3}$ (about 5 ft/mile), and with $Q$ and $p$ the same: $a \approx$ 5,000 ft, $b \approx$ 1,600 ft, and $c \approx$ 530 ft after 100 days and about 2,000 ft in 1,000 days. Inasmuch as the waste volume can move no farther in any direction than it does directly downgradient, in the latter case the limits would not have been approached within the time given and the volume would be essentially a cylinder of radius 530 ft after 100 days and almost 2,000 ft after 1,000 days of injection. In the former case the wastes would occupy essentially an elliptical cylinder having a major axis of some 18,300 ft and a minor axis of about 2,000 ft after 1,000 days.

Such distances as given cannot be expected to closely limit the distance to which radioactive contamination would spread because of the heterogeneity and anisotropy of the aquifers. The waste would be expected to move farther in the more permeable beds and less far in the beds less permeable than the average. Furthermore, if there is one direction in which ground water moves more easily than in another, the direction of flow inclines in this direction. For example, the elongation of the eastern ground-water mound (Figure 4) is caused by the upwarping of a two-mile wide belt of Ringold clays which rises above the water table near well 699-40-33 with a general northwesterly strike.[5] Consequently the water moves much more freely through the more permeable sands to the west and thus preferentially in northwestward and southeastward directions.

A waste contained in a stream of ground water moving from beneath disposal sites through unconsolidated granular deposits disperses both along and transverse to the direction of flow.[22] Dispersal in the direction of flow reduces the concentration of the contaminant if the waste is a slug temporarily introduced, and gives a warning at a locality downstream of the approach of a continuously introduced waste stream. Dispersal across the direction of flow spreads a contaminant more widely but reduces the concentration. The occurrence of hydraulic dispersion results in some portions of the waste traveling at velocities considerably exceeding the average.[23]

Such factors as heterogeneity, anisotropy, and dispersal assume great importance in determining the path of contaminants in the ground water. Consequently, the estimated mean lateral path of ground-water contamination shown in Figure 4 is taken to represent the probable minimum distance of travel from beneath disposal sites W and E to the Columbia River, R. Inasmuch as the construction of ground-water contours over much of the path shown was somewhat arbitrary due to scarcity of water-level data, there may be considerable deviation from the position of the path shown Furthermore, such contours will be shifted in response to continual changes in water levels in wells caused by fluctuations in river stage and the continuing artificial recharge of the aquifers by plant effluents. Nevertheless, based on the hydrologic conditions inferred in Figure 4, a minimum path from W to R of about 110,000 feet (about 21 miles) and from E to R of about 95,000 feet (18 miles) appears reasonable.

## Average Rates of Ground-Water Flow

The rate of ground-water flow is fixed by the vector quantity describing the maximum hydraulic gradient. Darcy's law for laminar flow is applicable but only enables the estimation of average velocities. Variation from the average is likely to be considerable, so that some small fraction of the flow may move at several times the average velocity, even in an essentially homogeneous formation. Rates of flow of tracers as much as three times the calculated average have been observed at Hanford (page 29), and Kaufman, et al[23] report that portions of a liquid may travel six or more times the average velocity even through essentially homogeneous porous media.

Interpretation of ground-water contour maps in terms of rates of ground-water flow is possible, if the transmissibility, and hence the permeability, of the aquifer is known. On the basis of measurements and estimates of aquifer characteristics previously presented, average velocities were calculated according to equation (4) ($v = \frac{P_f I}{7.48\, p}$). Furthermore, inasmuch as the permeability of both the glaciofluviatile and the Ringold aquifers ranged within relatively limited orders of magnitude (Table VIII), values for permeability were assumed for these aquifers at other well sites where quantitative data were not available. For example, an average permeability of 500 gpd/ft$^2$ was assumed for the Ringold aquifer, and an average permeability of 60,000 gpd/ft$^2$ was assumed for the glaciofluviatile sediments where they occur beneath the water table (see Figure 4). An effective porosity of 10 per cent was used in calculations except at sites where it had been more accurately determined.

The flow rates are shown on Figure 4. The map shows velocity vectors, with numerical values of average rate of movement in feet per day, as well as the quantity of water in gallons flowing in one day through a 1-mile-long section of the aquifer perpendicular to and bisected by the velocity vector for those sites where the transmissibility has been determined by pumping tests. The volume of flow to the southeast in the glaciofluviatile sediments is probably in the order of several hundred times that flowing eastward in the fine-grained Ringold sediments.

As mentioned previously, the average rate of flow is proportional to the hydraulic gradient under uniform conditions; but under the same gradient the quantity of flow is proportional to aquifer thickness and permeability Hence, eastward movement from disposal site W (Figure 4) occurs under an average gradient of about 20 ft/mile in the Ringold aquifer of permeability about 300 gpd/ft$^2$ (page 32). Average rates of movement are therefore only about 1-1/2 to 2 ft/day. Subsequent southeastward movement in the highly permeable glaciofluviatile sediments occurs chiefly under shallow gradients of only a few tenths of a foot per mile. Average velocities of about 7 ft/day were computed for most of this stretch. Direct eastward movement to the Columbia River through glaciofluviatile sediments is inhibited by the southern end of the eastern ground-water mound. Instead, general movement occurs more to the south through Ringold deposits under the influence of a moderate gradient of roughly 5 ft/mile, at an average velocity of 1 ft/day or less.

Travel Time

Based on the average ground-water velocities shown in Figure 4, a "travel time" of about 180 years is calculated for ground-water flow from W to R, and about 175 years from E to R. Significantly, the values and procedures adopted for calculating ground-water velocities result in conservative values for travel time. Furthermore, it must be emphasized that the maximum rate of movement of the ground water and even of some materials dissolved in it (e. g. , ruthenium-106 and nitrates) may be many times the average, while those dissolved constituents that enter into adsorption reactions (e. g. , strontium-90 and cesium-137) may move far slower than the water. Consequently, a more descriptive term than "travel time" is required for the actual behavior of concern. "Isotope travel time" is suggested inasmuch as several factors other than ground-water movement affect the available decay interval. These factors have been investigated in the laboratory and theoretically at the University of California and a summary of significant conclusions follows.[23, 24]

(a) Hydraulic phenomena produce velocity variations in laminar flow through homogeneous porous media that bring about a longitudinal mixing of selected intruding and displaced fluids. A diffuse zone or "concentration front" forms rather than a sharply defined interface. The depth of this zone increases in proportion to the distance traveled due to portions of the intruding contaminant moving at velocities exceeding the average.

(b) Ion exchange reactions may modify the propagation of a radio-contaminant in two ways: (1) the median velocity of the contaminant front will be predictably less than that of the liquid front, and (2) the depth or diffuseness of the front may be modified over that resulting from purely hydraulic phenomena. When the radio-contaminant is not selectively sorbed by the exchange medium, its front will become increasingly diffuse as it progresses through the medium. When the radiocontaminant has a selective affinity for the medium, as may be the case with strontium or cesium as the displacing cation, the front may not become more diffuse with distance but rather may tend to sharpen as propagation continues.

Empirical data obtained from radiologic monitoring of wells at Hanford have shown that the chemical form of $Ru^{106}$ in Hanford wastes is little affected by ion exchange, and anionic components of waste, such as nitrates, are apparently not affected at all. Sampling of the ground waters in more than 10 wells near western disposal sites ("W", Figure 4) disclosed that nitrate ion traveled about 0.6 ft/day whereas the $Ru^{106}$ from the same waste traveled at about 0.4 ft/day.[25] The average rate of ground-water movement at that time (1950) was calculated to be about 0.5 ft/day. In another case[25] radioruthenium moved southeastward about eight miles from an eastern disposal site (near "E", Figure 4) in less than one year at rates approaching 160 ft/day. Nitrate concentration increased to about 500 ppm in well 699-20-20 before trace concentrations of $Ru^{106}$ in the order of $10^{-7}$ $\mu c/cc$ appeared. This followed an approximately 18-month period of negligible supply of cooling water to the eastern swamp, during which the eastern mound subsided to the extent that a favorable hydraulic gradient existed from site E to well 699-20-20.[5] Subsequent tracer tests in this area (page 26) confirmed these high velocities.

## FUTURE STUDIES AND CONCLUSIONS

Additional geological and hydrological information is needed in waste disposal studies. As pointed out by Theis[22] and others in the field,[23] the techniques of ground-water studies used for estimating ground-water characteristics give average values - average velocity and average path of flow. In radioactive waste disposal the average is necessary as a starting point or a point of reference, but it is not good enough. For instance, the factors of heterogeneity and anisotropy of aquifers assume great importance in waste disposal. The important effects of such irregularities in the various geologic units upon the rate and direction of waste movement require that the geology be learned in great detail and that many wells be drilled to get these details. During 12 years of continuous well drilling at Hanford since the formal waste disposal research program began in 1947, 547 wells have been drilled for various purposes, totalling more than 107,000 feet. During the next 10 years it is contemplated that an average of about 11 wells per year totalling about 5,000 feet per year will be required for research purposes.[26] These wells are needed to (a) monitor any movement of ground-disposed radioactive solutions, (b) provide structures for hydrologic investigations permitting further evaluation of aquifer characteristics, (c) provide sediment samples for laboratory evaluation of ion exchange capacity, permeability, and mineral content, and to furnish soil column material for crib-life evaluations, and (d) provide basic geologic (stratigraphic) data. Items b, c, and d will provide data that permit predictions to be made of the probable behavior of wastes in the ground, and of the paths and rates of travel toward points of possible exposure. Item (a) will provide information on the actual behavior, information that can then be correlated to the data from which the predictions are made.

Because knowledge of the processes of dispersion is incomplete, this phenomenon must be further investigated in the field since the effective dispersion in the field is probably larger than indicated by laboratory experiments. A special 13-well field-scale facility for the study of dispersal phenomena during movement of ground water is under construction at the 699-62-43 site.

A geophysical seismic program is also being considered. It is expected that such a technique would improve forecasting of drilling needs, positioning of wells, completing of drilling projects at faster rates, and reducing overall drilling needs.[26]

In conclusion, a hydrogeological survey of waste-disposing sites must be extremely thorough. The current state of knowledge concerning aquifer characteristics and ground-water movement at Hanford has been presented herein. Estimates have been made of the aquifer volume a waste would occupy under certain stated conditions, the average rate at which the waste would travel, about where it would discharge into the Columbia River, and the approximate path it would take to get there. However, a considerable expansion of basic knowledge of ground-water movement and the geochemistry involved is required in order to insure that the geology and hydrology have been properly interpreted.[22]

## ACKNOWLEDGMENTS

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## APPENDIX I

### ANALYSIS OF MULTIPLE-WELL AQUIFER TEST

The nonequilibrium formula developed by Theis assumes that the aquifer is infinite, i. e., without boundaries or any change in its hydraulic characteristics. Methods are available for estimating the degree or manner in which an observed aquifer diverges from the idealized aquifer. As an example, the application of the image method in problems of ground-water hydraulics, as described by Ferris,[27] was applied to results obtained from a multiple-well pumping test at well site 699-31-53. Other Hanford examples have been described previously.[12]

The test well, 699-31-53, was pumped for eight hours at a rate of 605 gpm, and measurements were made of the drawdown and subsequent recovery of water level in an observation well located 51 feet to the south. Drawdown-recovery measurements in the observation well were made by an automatic water-stage recorder.

The log-log plot of the observed water-level data, s, against values of $r^2/t$ for the recovery phase is shown in Figure 5. For purposes of computation the graph of the observed data is superposed on the type curve which gives values of $W_{(u)}$ versus u, and with the coordinate axes of the two curves parallel, a position is found by trial for which most of the plotted points fall on the type curve. With the curves in this position, an arbitrary match point (MP) is chosen on one graph, and from the corresponding points on the other graph, values are derived for use in computing the coefficients of transmissibility and storage. By use of this method, these values were determined to be 108,000 gpd/ft and 0.06, respectively.

If a geologic or hydrologic boundary is not present, the extrapolated observed-data curve should correspond for its full length with the type curve. However, if a boundary is present within the effective radius of the area tested, the curve will deviate above or below the type curve. Figure 5

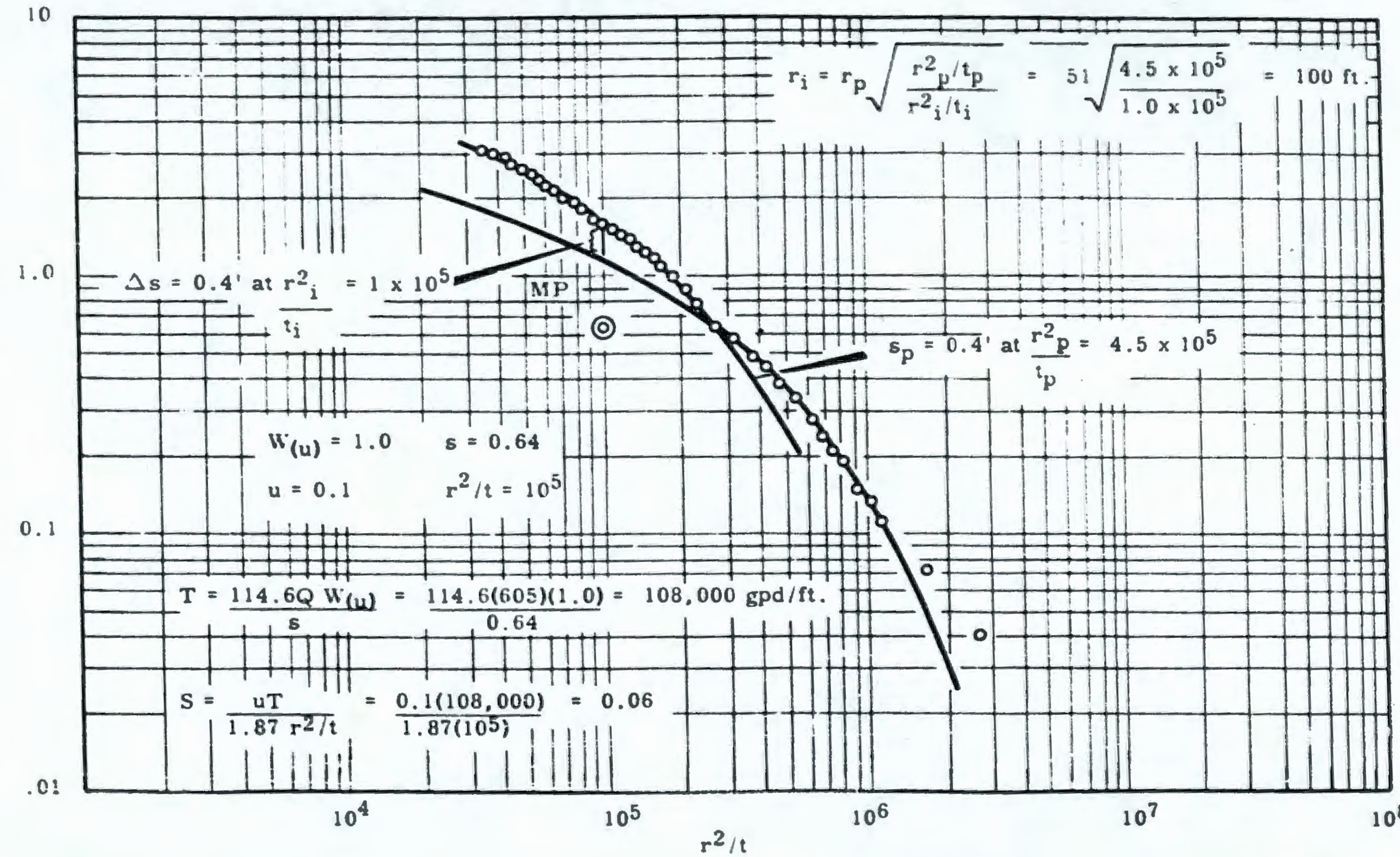



FIGURE 5

Logarithmic Graph of the Recovery of Water Level in Observation Well 699-31-53 shows that the observed data fall above the type curve, thus identifying a negative or discharging boundary. Using the formulas derived by Ferris, and knowing the distance from the observation well to the pumped well, $r_p$, and the time intercepts for both real $\left(\frac{r_p^2}{t_p}\right)$ and image $\left(\frac{r_i^2}{t_i}\right)$ wells at equal drawdown, the distance from the observation to the image well, $r_i$, is calculated to be about 100 feet. The distance to the negative boundary is therefore about 50 feet, and this boundary is interpreted as being caused by the difference in permeability between the glaciofluviatile sediments and the Ringold formation. The well is located on the edge of the glacial channel (see Figure 4), and below the water table it penetrates less than 10 feet of glaciofluviatile sediments before entering the Ringold.

# APPENDIX II

## EQUIPMENT

### Pumping System

A pumping system was developed by Raymond[28] that permits pumping of most of the project wells. The system includes:

(a) A Layne & Bowler 4-stage, 6 RM bowl assembly pump coupled to a 15 hp, 220-v, 60 cycle, 3-phase Franklin electric submersible motor. The unit is rated at 225 gpm at a total head of 200 feet.

(b) Ten-foot sections of Naylor spiral-weld 4-inch ID column pipe, with "Ever-Tite" quick-connect couplings.

(c) A trailer-mounted 30 KW G. M. diesel engine driven generator. Also on the trailer is a large diameter hand-cranked reel used for electrical cable storage, payout, and takeup.

(d) Surface piping consisting of 4-inch, schedule 40 pipe leading from the column pipe discharge elbow to 6-inch light weight irrigation line pipe. A 4-inch Sparling mainline meter measures water volume to an accuracy of 2 per cent and a 4-inch gate valve controls water flow.

(e) Hoisting equipment consisting of a 1/2-ton Muller gasoline engine-driven hoist and "A" frame mounted on the bed of a 2-1/2-ton 6 x 6 GMC truck and an Ingersoll-Rand air-driven winch.

Figure 6 shows the equipment in operation during the pumping test on well 199-F7-1.

### Shaped-Charge Perforating Equipment

With only a few exceptions, all the wells at Hanford have been perforated with the Mills' knife perforator. In order to perform non-routine reperforating where speed, positive and high density perforation, and use of light-weight hoisting equipment are desired, a shaped-charge system was

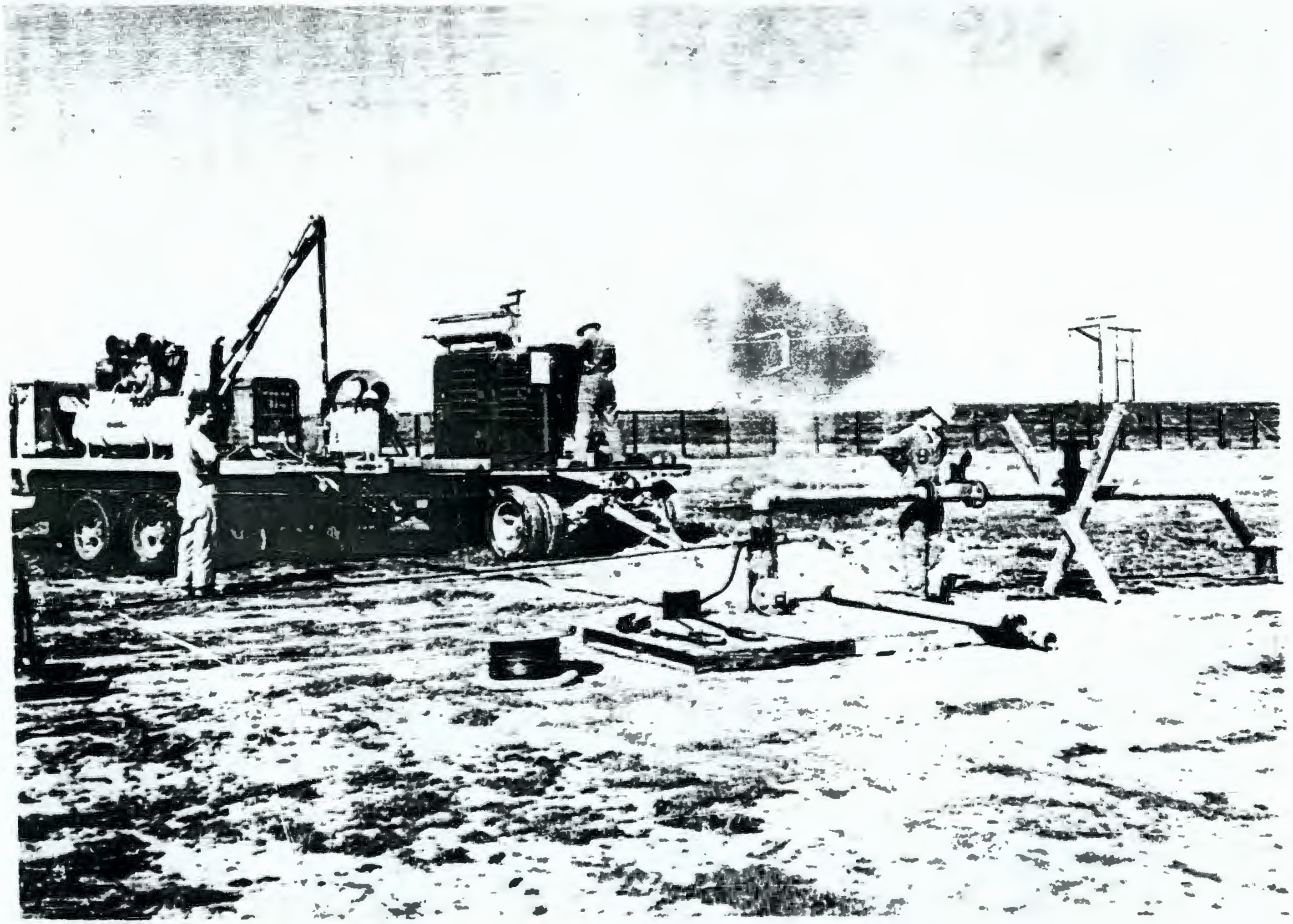


FIGURE 6

Photo Showing Pumping Equipment Installed at Pump-Test Site adapted by Raymond.[15] The shaped-charge perforating equipment consists of the shaped charges, primacord and detonators, carriers or guns, wire line, cable head, port plugs, primacord terminal unit, tandem connectors, end plugs, casing centralizers, various gaskets and "O" ring seals, and the blaster. The perforators are 10-gram charges of waxed RDX explosive pressed into steel containers. The carriers which locate and support the charges in the well are heavy-walled steel tubes with holes drilled through the wall and tapped to accommodate the charges. Gasketed aluminum plugs are screwed in against the charges to seal out water prior to detonation, and to retain the charges in position. Each carrier is 5 feet long, 5 inches in diameter, and will support 64 charges. Shot planes are at intervals of 3 inches along the carrier; each group of 4 shots is placed 45° further around the axis than the preceding four. Figure 7 shows an assembled carrier ready to be placed in a well.

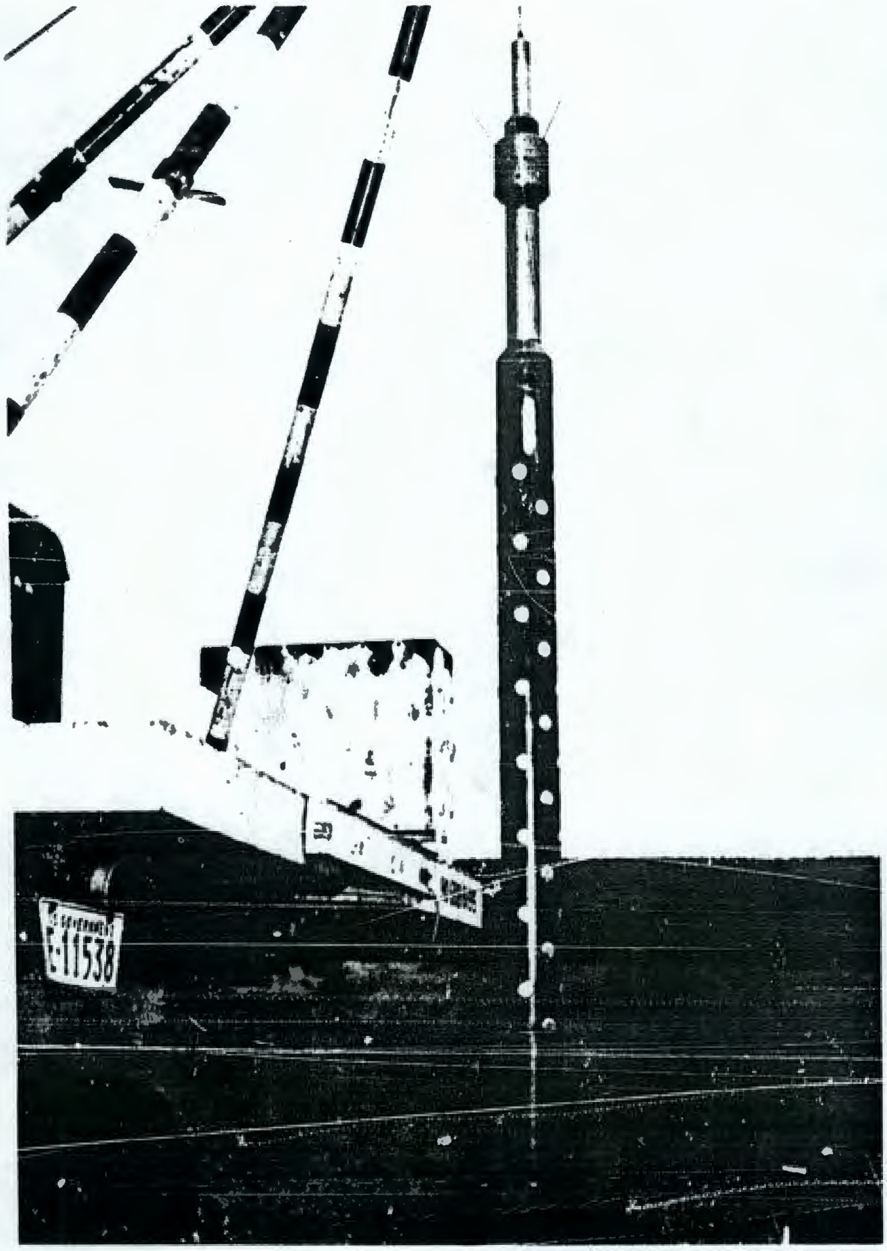



FIGURE 7

Photo Showing Assembled Shaped-Charge Carrier
for Perforating Wells

## APPENDIX III

### EVALUATION OF SINGLE-WELL AQUIFER TESTS

#### Modified Nonequilibrium Formula

Jacob[14] has shown that when plotted on semi-logarithmic paper, the theoretical drawdown curve approaches a straight line where sufficient time has lapsed after pumping started. This modified method should yield coefficients with accuracy comparable to the "type-curve" graphical solution of the Theis nonequilibrium formula if the data used are from the portion of the pumping test after the values of "u" in the following equation have become less than 0.01.

$$u = \frac{1.87\, r^2 S}{Tt} \qquad (9)$$

From the portion of the data which plots as a straight line on semi-logarithmic graph paper, the aquifer coefficients may be determined by use of the following equations:

$$T = \frac{264\, Q}{\Delta s} \qquad (10)$$

$$S = \frac{0.3\, T\, t_o}{r^2} \qquad (11)$$

T, Q, S, and r are as previously defined, and $\Delta s$ is the change in drawdown in feet per log cycle in the straight-line portion of the drawdown curve. The $t_o$ variable is the time value in days of the intercept of the straight-line portion of the curve (extended toward the starting line) and the zero drawdown line.

Equations (10) and (11) can be used also with data showing the recovery of water levels after pumping stops. In such a computation, "s" is the total recovery in feet and "t" is the time since pumping stopped. Values of T and S determined from recovery data should agree with those obtained from drawdown data. Recovery data have an advantage in that they are not affected by erratic pumping rates.

In general, it is not possible to determine the storage coefficient S from observations within the pumping well because the effective radius of the well is not known. In a well finished in unconsolidated materials, the water level in the pumped well is lower than the water level in an equivalent uncased hole by the amount of friction loss through the casing perforations or the well screen. If development of the well is incomplete, the packing of fine material in the formation adjacent to the well openings can greatly reduce the permeability and result in an effective radius which is considerably less than the nominal drilled size.

The modified nonequilibrium method of analysis is illustrated below (see "Test Procedure and Analysis of Test Data").

## Theis Recovery Formula

A useful corrolary to the nonequilibrium formula was derived by Theis[9] for the analysis of the recovery of a pumped well. If a well is pumped for a known period of time and then shut down and allowed to recover, the residual drawdown at any instant will be the same as if the discharge of the well had been continued but a recharge well with the same flow had been introduced at the same point at the instant the discharge stopped. For ordinary conditions of application the equation simplifies to

$$T = \frac{264\,Q}{s'} \log_{10} \frac{t}{t_{,}} \qquad (12)$$

where s' is the residual drawdown at any time during the recovery period (the difference between the observed water level and the non-pumping water level extrapolated fromt he observed trend prior to the pumping period); t is the time since pumping started, and $t_{,}$ is the time since pumping stopped.

When residual drawdown (or water level) is plotted on the linear scale of semi-logarithmic paper against the ratio $t/t_{,}$ to log scale, a straight line should result, the analysis proceeding in much the same manner as the modified nonequilibrium formula. Again, it is not possible to determine the coefficient of storage from the observations of the rate of recovery of a pumped well unless the effective radius is known.

Test Procedure and Analysis of Test Data

The data collected during the pumping test on well 699-26-15 are presented for analysis as they are representative of data obtained from the majority of tests conducted by personnel of the Geochemical and Geophysical Research Operation.

Well 699-26-15 was pumped for eight hours on July 1, 1958, and measurements of the drawdown of water level in the well were made frequently. The non-pumping or static water level stood about 55 feet below the top of the casing. The initial pumping rate of 260 gpm was reduced to 250 gpm after 35 minutes. Measurements of the recovery of water level were made frequently for five hours after pumping stopped and intermittently thereafter for a total of almost 17 hours. Figure 8 shows the drawdown-recovery hydrograph for the well. The data on which Figure 8 is based are included in Tables IX and X.

Figure 9 includes a time-drawdown curve and a time-recovery curve. The drawdown curve is obviously affected by the change in pumping rate. The recovery curve was obtained by plotting the amount the water level had raised from the extrapolated drawdown (see Figure 8) against the elapsed time after pumping ended. Theoretically, the recovery curve and the drawdown curve should coincide. If the pumping rate had remained exactly constant throughout the pumping period of the test, if the aquifer had been in exact hydraulic equilibrium before pumping began, and if all the assumptions of the non-equilibrium method were exactly true for this particular test, this would have occurred. However, these conditions are rarely completely met in the field, and the recovery curve will usually depart slightly from the drawdown curve.

Figure 10 includes a plot of residual drawdown versus the ratio $t/t_1$ and shows the resulting straight-line plot passing through the origin. A second curve shows the observed recovery of water levels after pumping stopped. This "modified" curve does not reflect the total recovery, hence the calculated transmissibility is somewhat higher than that obtained from the Theis recovery formula. The data from which the curves of Figure 10 were drawn are included in Tables IX and X.

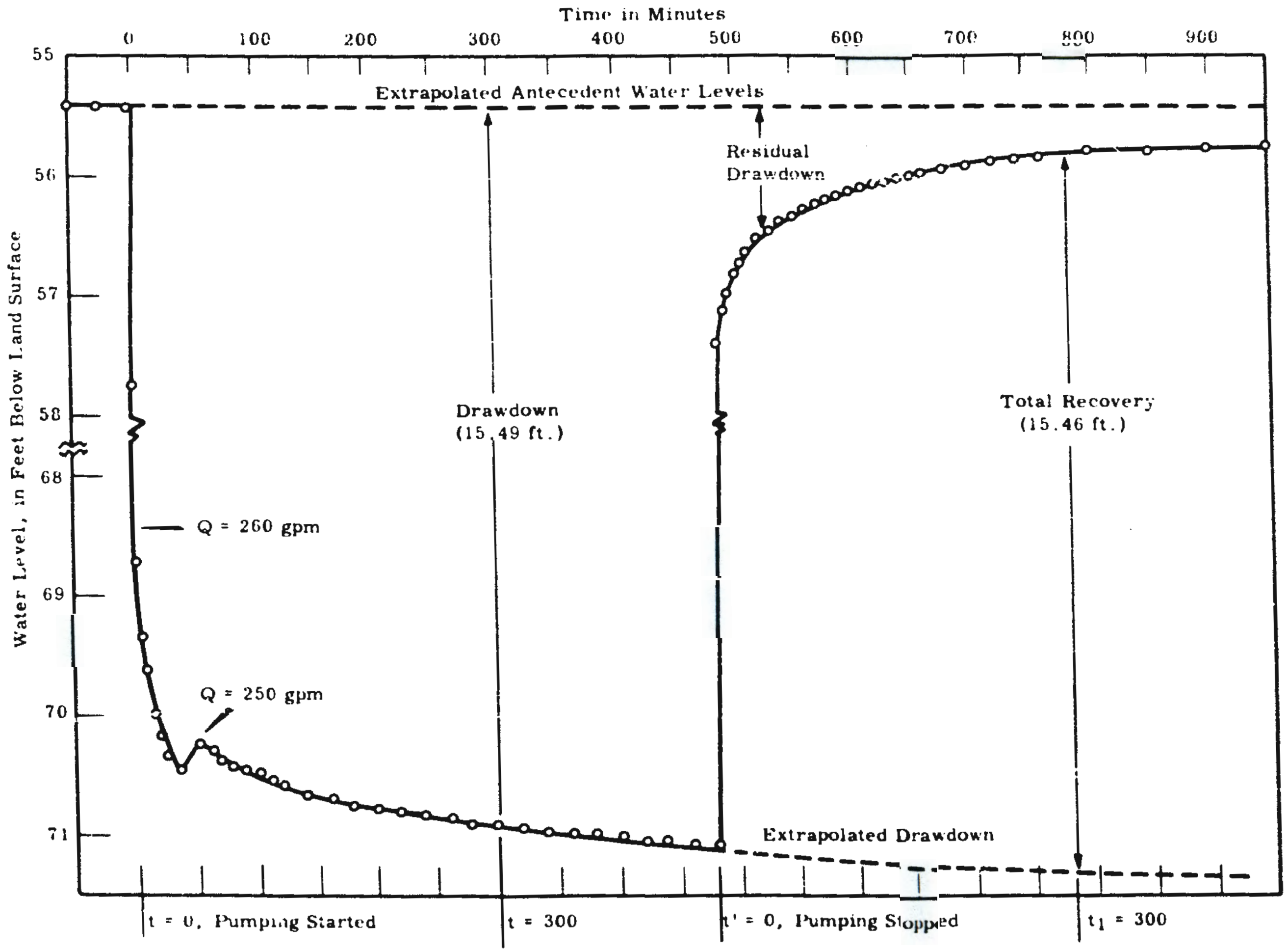



**FIGURE 8**

Hydrograph for Well 699-26-15

## TABLE IX

### DRAWDOWN OF WATER LEVEL IN PUMPED WELL 699-26-15
### AVERAGE PUMPING RATE Q = 252 GPM

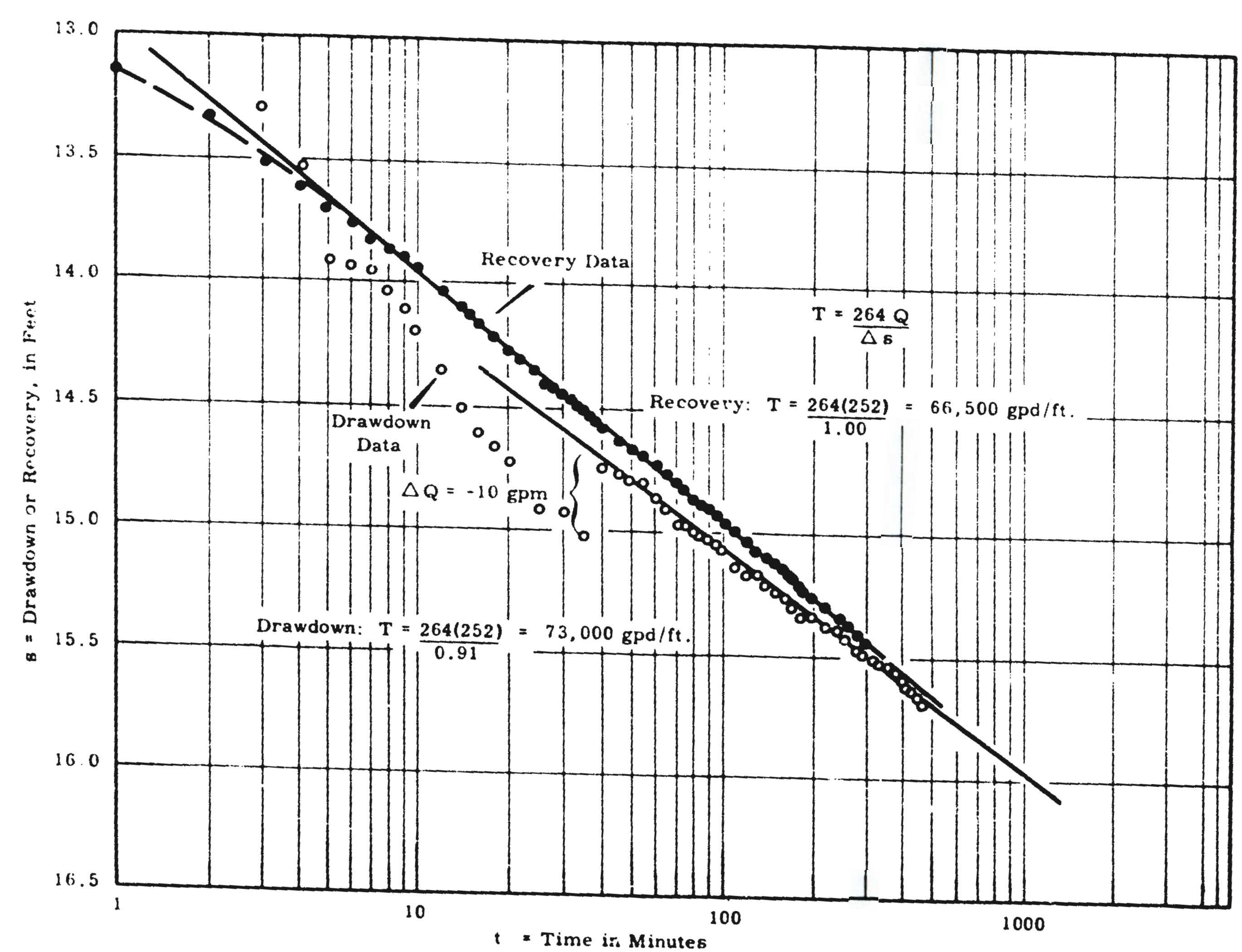
| Time Since Pumping Started t (minutes) | Observed Drawdown s (feet) | Time Since Pumping Started t (minutes) | Observed Drawdown s (feet) |
|---|---|---|---|
| | | 70 | 14.97 |
| 1 | 2.34 | 80 | 15.00 |
| 3 | 13.29 | 100 | 15.08 |
| 4 | 13.54 | 120 | 15.17 |
| 5 | 13.91 | 140 | 15.22 |
| 6 | 13.94 | 160 | 15.28 |
| 7 | 13.96 | 180 | 15.35 |
| 8 | 14.03 | 200 | 15.35 |
| 9 | 14.11 | 220 | 15.38 |
| 10 | 14.20 | 240 | 15.40 |
| 12 | 14.36 | 260 | 15.43 |
| 14 | 14.51 | 280 | 15.48 |
| 16 | 14.62 | 300 | 15.49 |
| 18 | 14.67 | 320 | 15.52 |
| 20 | 14.73 | 340 | 15.54 |
| 25 | 14.92 | 360 | 15.56 |
| 30 | 14.94 | 380 | 15.57 |
| 35 | 15.02 | 400 | 15.60 |
| 40 | 14.75 | 420 | 15.63 |
| 45 | 14.77 | 440 | 15.65 |
| 50 | 14.80 | 460 | 15.67 |
| 60 | 14.87 | 480 | 15.70 |

## TABLE X

### RECOVERY OF WATER LEVEL IN PUMPED WELL 699-26-15
### AVERAGE PUMPING RATE Q = 252 GPM

| Time Since Pumping Started t (minutes) | Time Since Pumping Stopped $t_1$ (minutes) | $t/t_1$ | Water Level (feet) | Residual Drawdown s' (feet) | Recovery s (feet) |
|---|---|---|---|---|---|
| 481 | 1 | 481 | 57.95 | 2.53 | 13.15 |
| 482 | 2 | 241 | 57.71 | 2.29 | 13.39 |
| 483 | 3 | 161 | 57.57 | 2.15 | 13.53 |
| 484 | 4 | 121 | 57.48 | 2.06 | 13.62 |
| 485 | 5 | 97 | 57.39 | 1.97 | 13.71 |
| 486 | 6 | 81 | 57.33 | 1.91 | 13.77 |
| 487 | 7 | 70 | 57.27 | 1.85 | 13.83 |
| 488 | 8 | 61 | 57.23 | 1.81 | 13.57 |
| 489 | 9 | 54 | 57.19 | 1.77 | 13.91 |
| 490 | 10 | 49 | 57.15 | 1.73 | 13.95 |
| 492 | 12 | 41 | 57.11 | 1.69 | 13.99 |
| 494 | 14 | 35 | 57.00 | 1.58 | 14.10 |
| 496 | 16 | 31 | 56.94 | 1.52 | 14.17 |
| 498 | 18 | 28 | 56.88 | 1.46 | 14.23 |
| 500 | 20 | 25 | 56.83 | 1.41 | 14.28 |
| 502 | 22 | 23 | 56.79 | 1.37 | 14.32 |
| 504 | 24 | 21 | 56.75 | 1.33 | 14.36 |
| 506 | 26 | 19.5 | 56.71 | 1.29 | 14.41 |
| 508 | 28 | 18.2 | 56.68 | 1.26 | 14.44 |
| 510 | 30 | 17.0 | 56.65 | 1.23 | 14.47 |
| 512 | 32 | 16.0 | 56.63 | 1.21 | 14.49 |
| 514 | 34 | 15.1 | 56.60 | 1.18 | 14.52 |
| 516 | 36 | 14.3 | 56.58 | 1.16 | 14.54 |

Table abbreviated for convenience; see Figure 8.




## FIGURE 9

Time - Drawdown (Recovery) Curves for Well 699-26-15

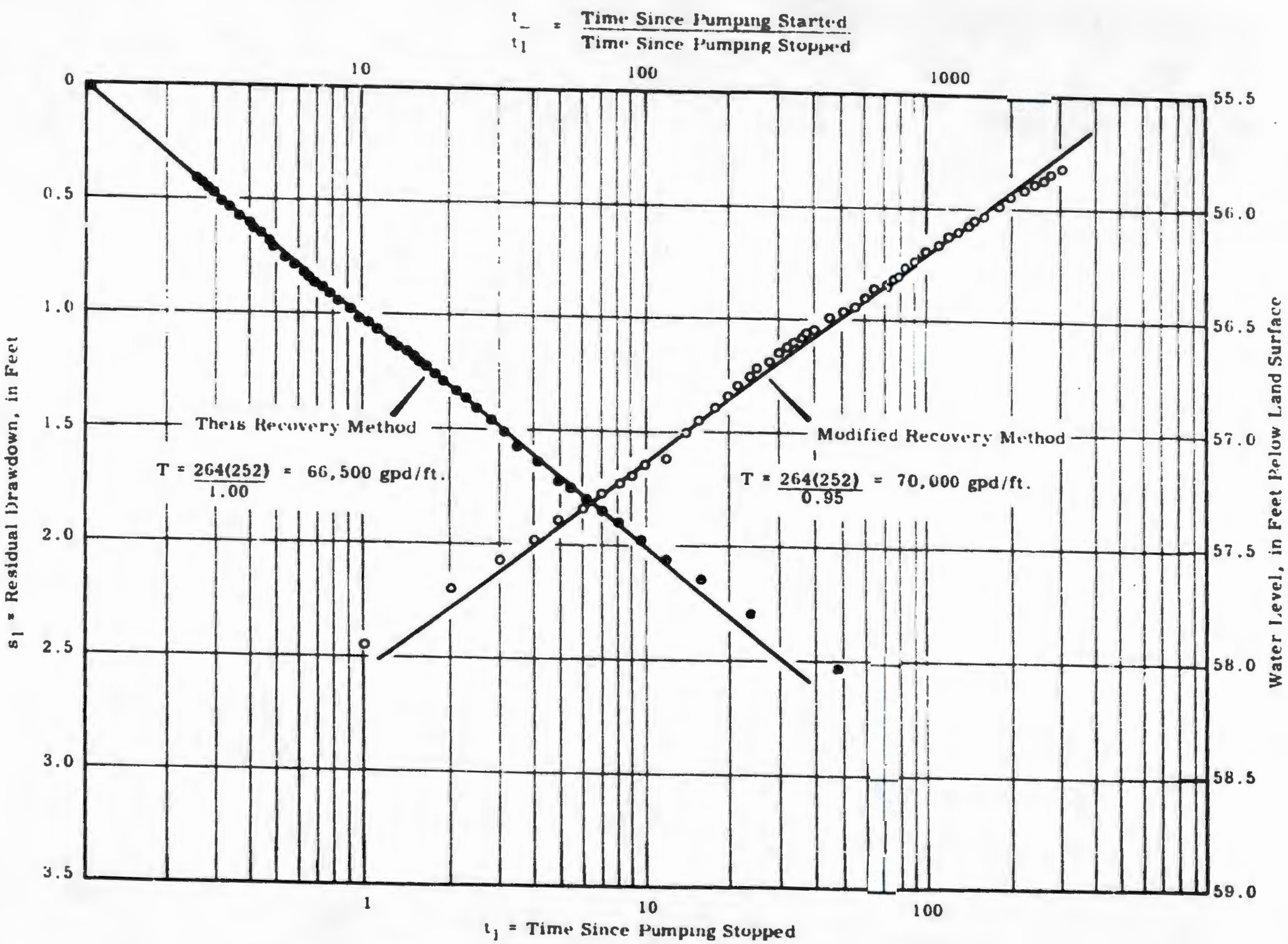



**FIGURE 10**

Recovery of Water Level in Well 699-26-15

A summary of the results obtained by the various graphical plots is as follows:

| Method | | Transmissibility gpd/ft | Comments on Application of Method to Data for this Test |
|---|---|---|---|
| Modified nonequilibrium | drawdown | 73,000 | Affected by changes in pumping rate. |
| | recovery | 66,500 | Extrapolation of water-level curve may involve possible errors. |
| Theis | recovery | 66,500 | Yields maximum return for data available. |
| | modified | 70,000 | Measurements not based on total recovery. Provides good field check. |

It appears that the more reliable data give a field coefficient of transmissibility of about 67,000 gpd/ft. According to the driller, well 699-26-15 penetrated 75 feet of gravel and then 25 feet of cemented sand and gravel, with a static water level of 55 feet. Below this are beds of clay with some silt, sand, and gravel in the lower parts with basalt at 350 feet. It is estimated that the effective saturated thickness of the tested aquifer is about 45 feet and that the aquifer consists of both glaciofluviatile and Ringold deposits. The apparent average permeability of these deposits is consequently about 1,500 gpd/ft$^2$.

## Adjustment of Test Data for Thin Aquifers

One of the basic assumptions of the Theis formula is the stipulation of a constant value of transmissibility. However, under water-table conditions, the drawdown of water level by a discharging well reduces the saturated thickness of the aquifer, and, if this reduction is appreciable, the transmissibility is not constant but decreases with time. According to Jacob[29] if the observed drawdowns are adjusted (reduced) by the factor $s^2/2m$, where "s" is the observed drawdown and "m" is the saturated thickness, the value of transmissibility will correspond to equivalent confined flow of uniform depth. Such compensation of the drawdown data should result in a straight-line graph for the semilog plottings of the modified nonequilibrium method.

Water-level measurements obtained during an eight-hour drawdown-recovery test on well 199-F7-1 were adjusted by the factor $s^2/2m$. Static water stood at a depth of about 9 feet, and the driller reported 20 feet of gravel and cobbles, 4 feet of small gravel, and then 125 feet of clay with thin interbeds of silt and sand. Thus the saturated thickness of the effective aquifer was about 15 feet. At an average pumping rate of 107 gpm a drawdown of 7.3 feet was reached after eight hours. Figure 11 is a graphical representation of the observed drawdown measurements and of the adjusted drawdown data. If the test data are not reduced by $s^2/2m$ for the thin 15-foot aquifer, the transmissibility calculates as 33,000 gpd/ft and the field coefficient of permeability as 2,200 gpd/ft$^2$. As calculated from the adjusted data, however, the respective values are 59,000 gpd/ft and 3,900 gpd/ft$^2$.

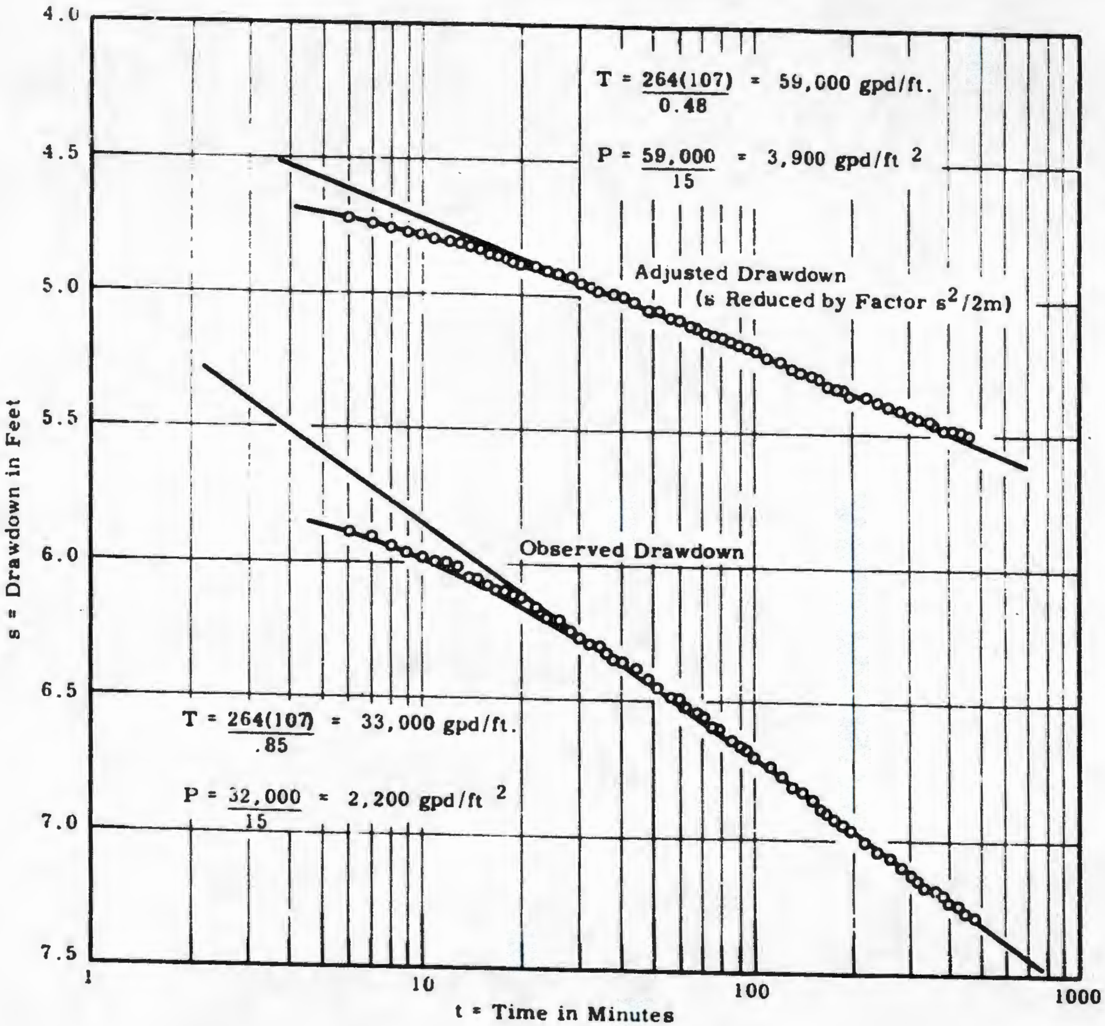



**FIGURE 11**

Time - Drawdown Curves for Well 199-F7-1

## APPENDIX IV

### ESTIMATING TRANSMISSIBILITY FROM SPECIFIC CAPACITY

An examination of the relationship between specific capacity and transmissibility is warranted because even a rough figure for T is of value in many areas and because of the general difficulty is ascertaining its exact value without more elaborate aquifer tests. In 1941, Theis[30] developed a relationship theoretically exact within the limits of necessary idealized assumptions, and modified the theoretical formula empirically to data generally readily available in the field. The Theis equation for water-table wells having diameters of about one foot in unconsolidated sediments is as follows:

$$T' = \frac{Q}{s}(1 \pm 0.3)(1300 - 264 \log 5S + 264 \log t) \qquad (13)$$

where $T' = T - (\frac{264\,Q}{s} \log T \times 10^{-5})$ and the terms, T, Q, s, S, and t are, respectively, the coefficient of transmissibility, discharge, drawdown, coefficient of storage, and time of pumping. The factor $(1 \pm 0.3)$ should be adjusted upward to (1.3) for small or poorly developed wells or those with poorly perforated casing, and downward to (0.7) for larger and well-developed wells. It is obvious that T cannot be determined from the computed values of T', although charts can be drawn giving the values of T' for various values of T and Q/s, and that from such charts, knowing T' and Q/s, the value of T can be ascertained. However, it soon becomes apparent that large changes in S or in t correspond to relatively small changes in the coefficient of transmissibility and specific capacity. Consequently, equation (13) may be simplified to the approximation

$$T = \frac{Q}{s}(1700) \qquad (14)$$

which constitutes a useful means of estimating the general order of magnitude of the transmissibility of water-table aquifers.

The graph in Figure 12 was prepared by computing the theoretical drawdown, using the Theis nonequilibrium formula, in 8-inch diameter wells for various coefficients of transmissibility and storage, computing the specific capacity (assuming wells having 100 per cent efficiency), and plotting the specific capacity against transmissibility. The resulting lines cover a range of S = 0.05 to 0.20 and a pumping period t = 8 to 24 hours. These variables are those most encountered at Hanford. The graph may be used to determine the approximate coefficient of transmissibility of an aquifer in an area where the specific capacities of wells are known. Furthermore, determination of the approximate specific capacity of a proposed well ending in an aquifer having known hydraulic characteristics is often helpful for water well planning purposes. Although determinations made from the graph may not be exact, owing to unknown factors that must be estimated, nevertheless the graph serves as a "yardstick" for approximation as well as for illustration of the relationship between specific capacity and transmissibility. Values for T estimated from the graph are in the same order of magnitude as those estimated from equation (14).

The dashed line in Figure 12 is the least squares linear estimation line of the plotted points "x". These points are the values for transmissibility and specific capacity derived from controlled aquifer tests (Table III). A comparison of the theoretical specific capacity lines, based on 100 per cent efficiency, with the specific capacity determined by field test, indicates that on the average the wells pumped at Hanford are only about 25-35 per cent efficient.

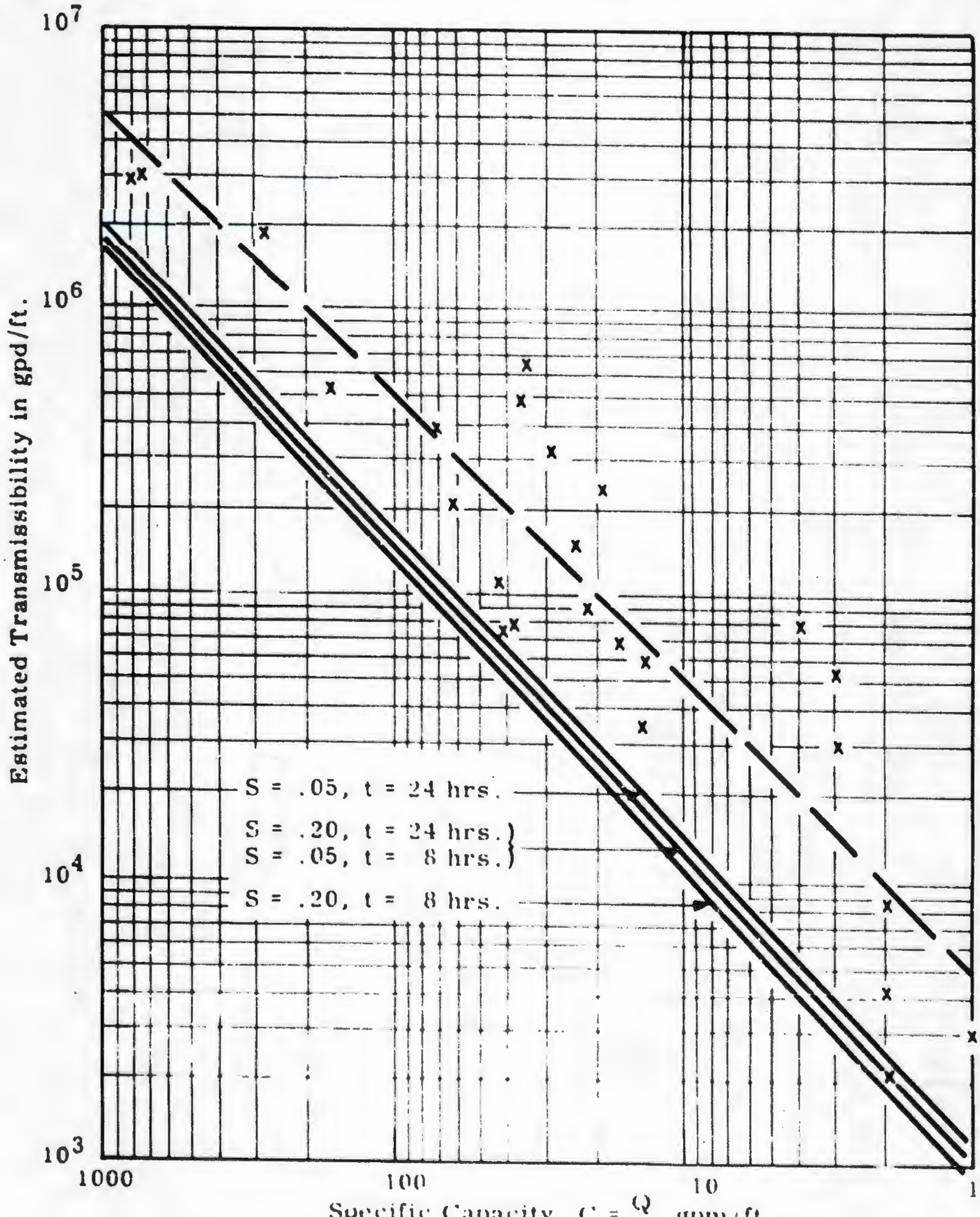



FIGURE 12

Relationship Between Specific Capacity and Estimated Transmissibility

## APPENDIX V

### STEP-DRAWDOWN TESTS

The drawdown in a well resulting from the withdrawal of water is made up of (a) head loss resulting from laminar flow in the formation, and (b) head loss resulting from turbulent flow in the zone outside the well, through the well openings, and in the well casing. Jacob[14] presents a method for evaluating the turbulent head losses using the equation.

$$s_w = BQ + CQ^2 \qquad (15)$$

where $s_w$ is the drawdown in the pumped well, B is the "formation-loss" constant, or the "resistance" of the formation, C is the "well-loss" constant, and Q is the discharge. Rorabaugh[31] derived a similar equation and presents a more exact method for evaluation when a larger range of pumping rates is encountered. In practice, however, equation (15) has been found to be entirely adequate for most engineering applications.

Data collected during a step-drawdown test on well 699-55-50 #1 are presented for illustration. Well #1 is an 8-inch perforated well with 45 feet of perforations spaced 6 holes per round and 2 rounds per foot. Figure 13 is a plot of the test data for well 699-55-50 #1. It can be noted that the recession curve at 105 gpm had a "slope" of 0.015 feet per log cycle. The slope at 205 gpm was estimated by multiplying the slope at 105 gpm by the ratio of 205/105 giving 0.029 feet per log cycle. The slope at 360 gpm was 0.052 feet per log cycle. These slopes were used to extrapolate each step of the test beyond the period of pumping of each step as shown by the dashed lines in Figure 13. These extrapolations were used to obtain the incremental drawdown caused by a change in pumping rate. When the test began, the pumping rate immediately increased from zero to 105 gpm. After 45 minutes of pumping the drawdown was 0.13 feet, and 45 minutes after pumping rate had been increased to 205 gpm the incremental drawdown caused by this

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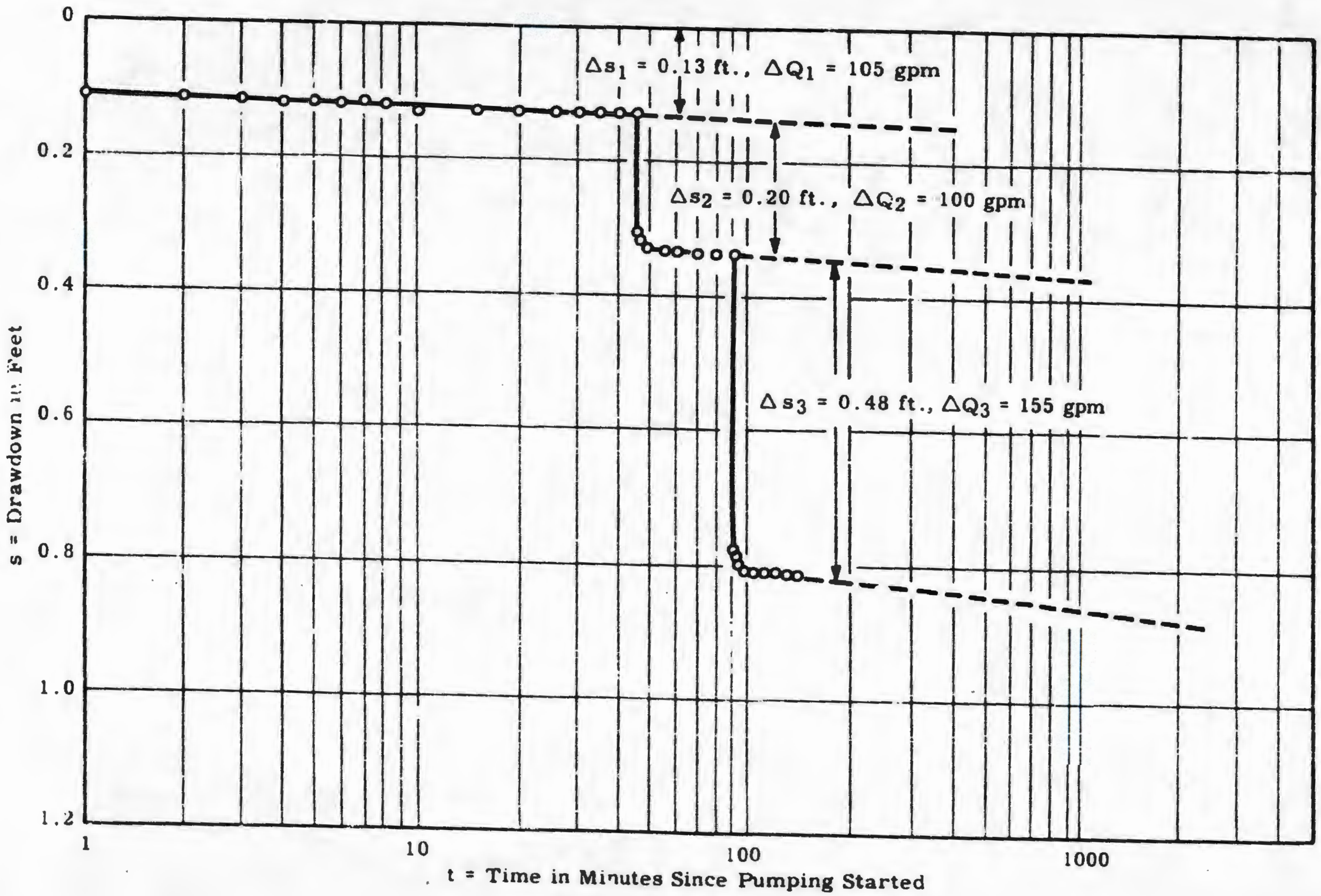


**FIGURE 13**

Time - Drawdown Curves Obtained During Step-Drawdown Test on Well 699-55-50 #1





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100 gpm increase was 0.20 feet. The third incremental drawdown of 0.48 feet resulted from a 155 gpm increase of pumping rate to 360 gpm. (Note: customarily, the periods of pumping are of one hour's duration.)

The step-drawdown calculations are arranged in tabular form shown in Table XI.

TABLE XI

STEP-DRAWDOWN CALCULATIONS

| Step | Q Pumping Rate (gpm) | $\Delta s$ Incremental Drawdown (feet) | $s_w$ Drawdown at Q (feet) | $s_w/Q$ (feet/gpm) |
|---|---|---|---|---|
| 1 | 105 | 0.13 | 0.13 | 0.00124 |
| 2 | 205 | 0.20 | 0.33 | 0.00161 |
| 3 | 360 | 0.48 | 0.81 | 0.00225 |

The values $s_w/Q$ and Q are plotted on arithmetic coordinate paper as shown in Figure 14. From the line through the points of Figure 14 the following equation was determined: $s_w = 0.00082\,Q + 0.0000040\,Q^2$, which is the form of equation (15) and is the approximate equation for the drawdown in well 699-55-50 #1 for the stated pumping period. Figure 15 shows a plot of this equation and the observed drawdowns for several pumping rates.

The plotted points are taken from the following table:

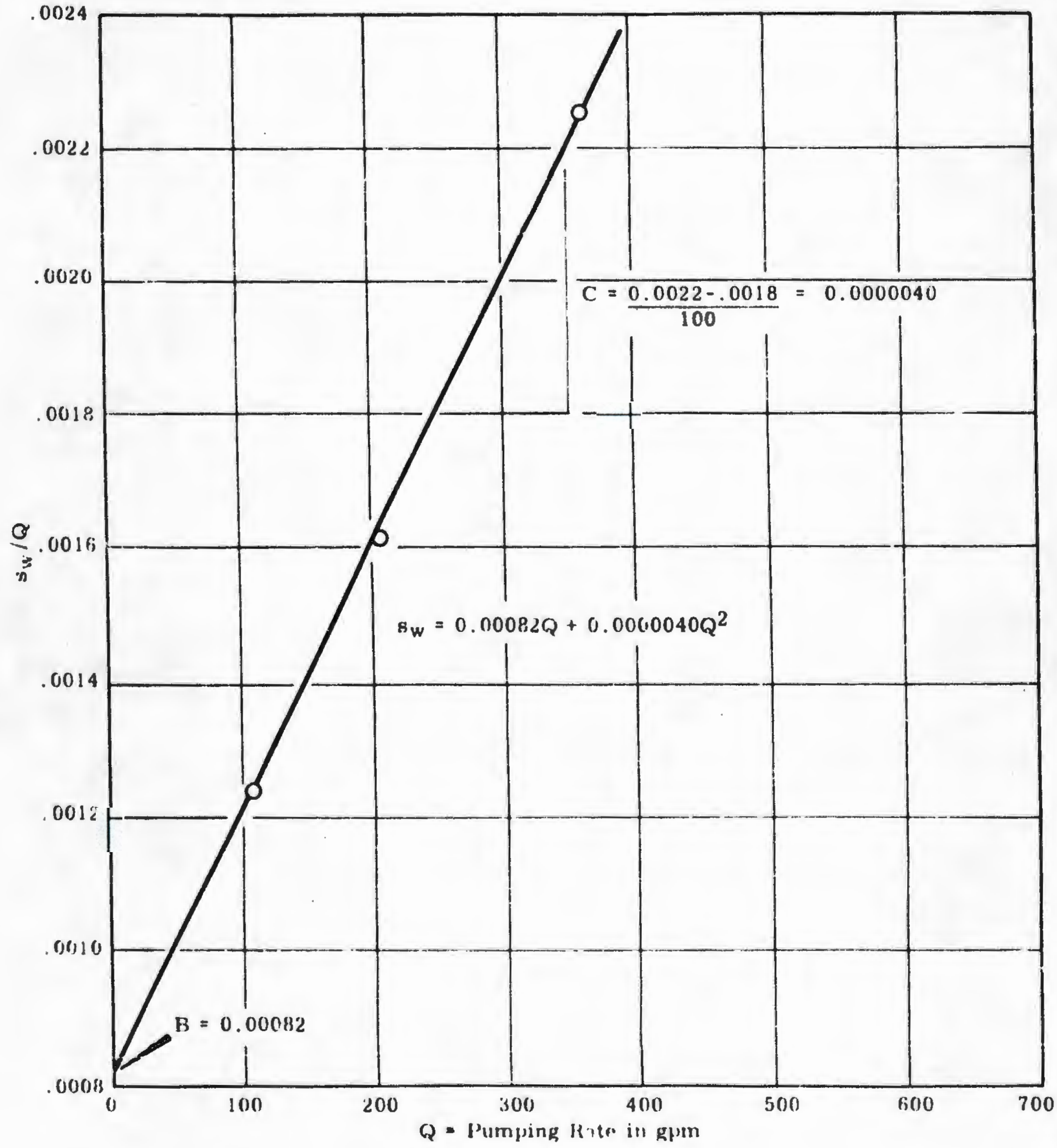



FIGURE 14

Plot of $\frac{s_w}{Q}$ vs Q to Solve for Values of B and C

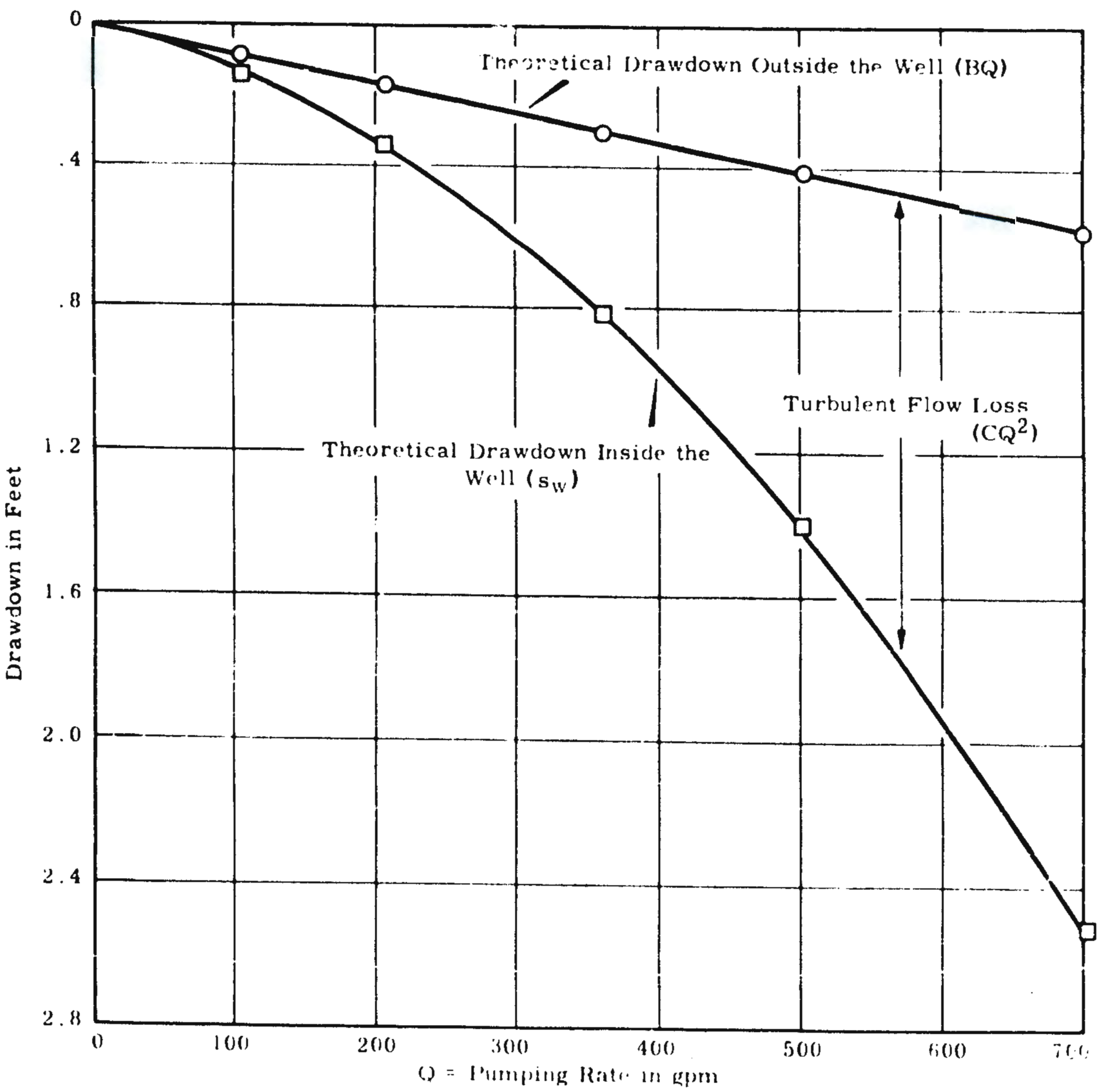



FIGURE 15

Drawdown - Yield Curve for Well 699-55-50 #1

## TABLE XII

## CALCULATIONS FOR DRAWDOWN-YIELD CURVE

$$s_w = BQ + CQ^2$$

$$s_w = 0.00082\ Q + 0.0000040\ Q^2$$

| Q Pumping Rate (gpm) | BQ Formation Loss (feet) | $CQ^2$ Well Loss (feet) | $s_w$ Theoretical Drawdown (feet) | $BQ/s_w$ Well Efficiency |
|---|---|---|---|---|
| 105 | 0.09 | 0.04 | 0.13 | 0.70 |
| 205 | 0.17 | 0.17 | 0.34 | 0.50 |
| 360 | 0.30 | 0.52 | 0.82 | 0.37 |
| 500 | 0.41 | 1.00 | 1.41 | 0.29 |
| 700 | 0.57 | 1.96 | 2.53 | 0.23 |

A quick estimate of the well-loss constant C may be obtained by solving the equation

$$C = \frac{\dfrac{\Delta s_3}{\Delta Q_3} - \dfrac{\Delta s_2}{\Delta Q_2}}{\Delta Q_3 + \Delta Q_2} \qquad (16)$$

where the $\Delta s$ terms are the incremental drawdowns obtained at successive increases in pumping rate $\Delta Q$. From Figure 13 and Table XI, where $\Delta s_3 = 0.48$, $\Delta s_2 = 0.20$, $\Delta Q_3 = 155$, and $\Delta Q_2 = 100$:

$$C = \frac{\dfrac{0.48}{155} - \dfrac{0.20}{100}}{155 + 100} = \frac{0.0031 - 0.0020}{255} = 0.0000043$$

Such an estimate is of value for multiple-step-drawdown tests of very short pumping periods or where turbulence of the water in the well precludes continued accurate water-level measurements.

## APPENDIX VI

### ESTIMATING TRANSMISSIBILITY FROM CYCLIC FLUCTUATION DATA

Ferris[32] has shown that the equation for the range of ground-water fluctuation in an observation well of known distance from the aquifer contact with the surface-water body, whose stage changes sinusoidally, has the non-dimensional form:

$$s_r = 2\,s_o\,e^{-4.8\,X\sqrt{\frac{S}{t_o T}}} \qquad (17)$$

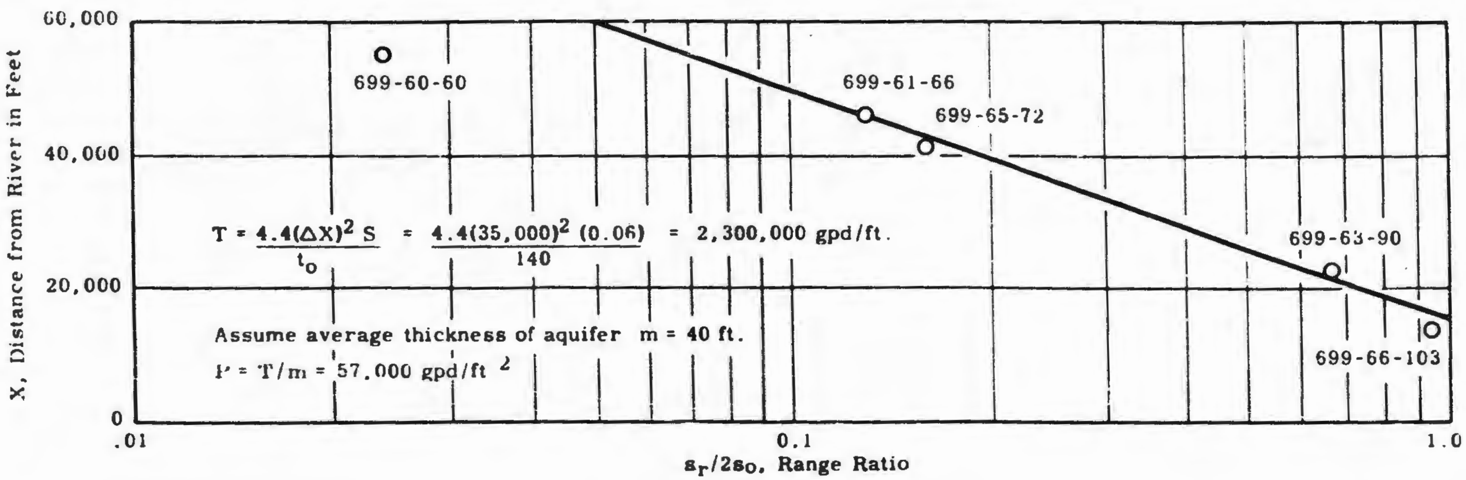
where

$s_r$ = range in ground-water stage, in feet,
$s_o$ = amplitude or half range of river stage, in feet,
$X$ = distance from the observation well to the surface-water contact with the aquifer ("suboutcrop"), in feet,
$t_o$ = period of the stage fluctuation, in days,
$S$ = coefficient of storage,
$T$ = coefficient of transmissibility, gpd/ft.

For convenience equation (17) can be written:

$$2.1\sqrt{\frac{S}{t_o T}} = \frac{-\log_{10}\left(\frac{s_r}{2\,s_o}\right)}{X} \qquad (18)$$

The right-hand member of equation (18) may be represented as a slope by plotting on semilog paper the logarithm of the average range ratio ($s_r/2s_o$) for each well against the respective distance (X) of each well from the river. If the change in logarithm of the range ratio is selected over one log cycle, the numerator of this slope expression reduces to unity. Thus, equation (18) may be reduced to $T = 4.4\,(\Delta X)^2\,S/t_o$. Figure 16 roughly illustrates this method, and shows the plotted points for five wells which are located in the eastward trending glaciofluviatile channel north of Gable




## FIGURE 16

Semilog Plot of the Ratio of Ground-Water Stage to Stream Stage vs. Distance from Well to Cyclic Stream

Butte (for well location, see map, Figure 1). The data for well 699-60-60 were discounted because the hydrograph of the well indicates that the water level is influenced also by artificial recharge which masks the ground-water range due to the influence of the river.

Table XIII includes the data from which estimates of transmissibility were made for the aquifers penetrated by 15 wells in which the water level fluctuated in response to changes in Columbia River stage. The range in ground-water stage ($s_r$) was averaged for the period of record as was the range of river stage ($2\,s_o$). Inasmuch as the river fluctuation is not strictly sinusoidal but generally occurs as a single sharp crest each year, the period of the river fluctuation ($t_o$) was taken as an average of 140 days. As indicated by preceding equations, it is necessary that the coefficient of storage S be known in order to evaluate T. Only a few data are available giving values for S at Hanford, but where it has been calculated,[6, 12, 18] a range within 0.06 to 0.10 appears reasonable.

The indicated values (Table XIII) of the coefficient of transmissibility should be considered tentative. However, these data serve to demonstrate the applicability of the method described for analyzing cyclic fluctuations of ground-water level. The results, except for several inordinately large values, appear to be within the correct order of magnitude of transmissibility as derived previously for sites elsewhere on the project. The estimates of permeability were made assuming various effective thicknesses for the aquifers.

## TABLE XIII

### TRANSMISSIBILITY COEFFICIENTS ESTIMATED FROM CYCLIC FLUCTUATION DATA

| Well Number | Years of Record | Average Ground-Water Range $s_r$ (feet) | Distance from River (feet) | Average Range Ratio ($s_r/2s_o$) | Transmissibility (gpd/ft) when S = 0.06 | Transmissibility (gpd/ft) when S = 0.10 | Estimated Permeability (gpd/ft$^2$) when S = 0.06 | Estimated Permeability (gpd/ft$^2$) when S = 0.10 |
|---|---|---|---|---|---|---|---|---|
| 699-60-60 | 6 | 0.45 | 55,000 | 0.024 | | | | |
| -61-66 | 3 | 2.40 | 46,000 | 0.13 | 2,300,000 | 3,800,000 | 57,000 | 95,000 |
| -65-72 | 8 | 3.00 | 41,000 | 0.16 | | | | |
| -63-90 | 9 | 12.75 | 23,000 | 0.67 | | | | |
| -66-103 | 6 | 17.80 | 14,000 | 0.94 | | | | |
| -57-29 | 12 | 4.44 | 10,000 | 0.28 | 610,000 | 1,000,000 | 17,000 | 29,000 |
| -62-32 | 12 | 4.62 | 11,000 | 0.29 | 790,000 | 1,300,000 | 23,000 | 37,000 |
| -63-25 | 3 | 3.70 | 4,000 | 0.24 | 80,000 | 130,000 | 1,100 | 1,700 |
| -67-77 | 12 | 3.79 | 5,500 | 0.20 | 115,000 | 190,000 | 960 | 1,600 |
| -70-68 | 4 | 2.99 | 7,000 | 0.16 | 145,000 | 240,000 | 1,200 | 2,000 |
| -71-84 | 12 | 6.62 | 1,000 | 0.35 | 9,000 | 15,000 | 90 | 150 |
| -72-86 | 12 | 10.79 | 1,000 | 0.57 | 31,000 | 51,000 | 310 | 510 |
| -92-38 | 9 | 6.52 | 1,200 | 0.42 | 19,000 | 32,000 | 190 | 320 |
| -97-48 | 12 | 3.82 | 4,000 | 0.23 | 74,000 | 120,000 | 500 | 800 |
| -HAN-23 | 12 | 4.50 | 5,000 | 0.28 | 155,000 | 260,000 | 3,700 | 6,200 |
| | 3 | (from ref. 18) | | | 520,000 | 860,000 | 6,100 | 10,000 |

| Columbia River at Area | Years of Record | Average River-Stage Range $2s_o$ (feet) |
|---|---|---|
| B | 12 | 19.1 |
| D | 12 | 15.9 |
| H | 12 | 15.6 |
| F | 12 | 16.0 |





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